PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

VOL. 68 JUNE, 1942 No. 6

TECHNICAL PAPERS

AND

DISCUSSIONS

Published monthly, except July and August, at Prince and Lemon Streets, Lancaster, Pa., by the American Society of Civil Engineers. Editorial and General Offices at 33 West Thirty-ninth Street, New York, N. Y. Reprints from this publication may be made on condition that the full title of Paper, name of Author, page reference, and date of publication by the Society, are given.

Entered as Second-Class Matter, September 23, 1937, at the Post Office at Lancaster, Pa., under the Act of March 3, 1879. Acceptance for mailing at special rate of postage provided for in Section 1103, Act of October 3, 1917, authorized on July 5, 1918.

Subscription (if entered before January 1) $8.00 per annum. Price $1.00 per copy

Copyright, 1942, *by the* AMERICAN SOCIETY OF CIVIL ENGINEERS
Printed in the United States of America

CURRENT PAPERS AND DISCUSSIONS

		Discussion
Salts in Irrigation Water. *Raymond A. Hill*	June, 1941	
Discussion	Sept., Oct., Dec., 1941	Closed*
Pipe-Line Flow of Solids in Suspension: A Symposium	Oct., 1941	
Discussion	Oct., Dec., 1941, Mar., Apr., 1942	Closed*
Protective and Remedial Measures for Sanitary and Public Health Engineering Services: Progress Report of the Sanitary and Public Health Engineering Division of the National Committee of the Society, on Civilian Protection in War Time	Jan., 1942	
Discussion	Mar., Apr., June, 1942	Closed
Fundamental Aspects of the Depreciation Problem: A Symposium	Nov., 1941	
Discussion	Nov., Dec., 1941, Feb., Mar., Apr., June, 1942	July 1, 1942
Stability of Granular Materials. *R. E. Glover and F. E. Cornwell*	Nov., 1941	
Discussion	Nov., 1941, Mar., Apr., 1942	July 1, 1942
Timber Friction Pile Foundations. *Frank M. Masters*	Nov., 1941	
Discussion	Feb., Apr., May, June, 1942	July 1, 1942
Bending Moments in the Walls of Rectangular Tanks. *Dana Young*	Nov., 1941	
Discussion	Feb., 1942	July 1, 1942
Energy Loss at the Base of a Free Overfall. *Walter L. Moore*	Nov., 1941	
Discussion	Jan., Apr., June, 1942	July 1, 1942
Design of St. Georges Tied Arch Span. *J. M. Garrelts*	Dec., 1941	
Discussion	Feb., Apr., May, June, 1942	July 1, 1942
Allocation of the Tennessee Valley Authority Projects. *Theodore B. Parker*	Dec., 1941	
Discussion	Feb., Mar., May, June, 1942	July 1, 1942
Development of Transportation in the United States. *J. E. Teal*	Dec., 1941	
Discussion	Feb., Mar., Apr., May, 1942	July 1, 1942
Lateral Stability of Unsymmetrical I-Beams and Trusses in Bending. *George Winter*	Dec., 1941	
Discussion	Feb., Mar., 1942	July 1, 1942
Drainage of Leveed Areas in Mountainous Valleys. *Gordon R. Williams*	Jan., 1942	
Discussion	Jan., Apr., May, June, 1942	July 1, 1942
Hydraulic Design of Drop Structures for Gully Control. *B. T. Morris and D. C. Johnson*	Jan., 1942	
Discussion	Apr., May, June, 1942	July 1, 1942
Analytical and Experimental Methods in Engineering Seismology. *M. A. Biot*	Jan., 1942	
Discussion	Apr., May, June, 1942	July 1, 1942
Effect of Variation of Elastic Characteristics on Static Unknowns. *A. Hrennikoff*	Jan., 1942	
Discussion	Apr., May, 1942	July 1, 1942
Design of Sign Letter Sizes. *Adolphus Mitchell and T. W. Forbes*	Jan., 1942	
Discussion	May, June, 1942	July 1, 1942
The Grease Problem in Sewage Treatment. *Almon L. Fales and Samuel A. Greeley*	Feb., 1942	
Discussion	Feb., May, June, 1942	July 1, 1942
Unusual Events and Their Relation to Federal Water Policies. *W. G. Hoyt*	Feb., 1942	
Discussion	Apr., 1942	July 1, 1942
Reduction of Mineral Content in Water with Organic Zeolites. *R. F. Goudey*	Feb., 1942	
Discussion	Feb., Apr., May, 1942	July 1, 1942
Resistance of Soft Fill to Static Wheel Loads. *W. Watters Pagon*	Feb., 1942	July 1, 1942
Technique of Determining Shearing Strength of Soils: Progress Report of a Special Committee of the Soil Mechanics and Foundations Division on the Technique of Soil Tests	Feb., 1942	
Discussion	Feb., Apr., June, 1942	July 1, 1942
Viscosity and Surface Tension Effects on V-Notch Weir Coefficients. *Arno T. Lenz*	Mar., 1942	
Discussion	June, 1942	Aug. 1, 1942
Hydrodynamics of Model Storm Sewer Inlets Applied to Design. *G. S. Tapley*	Mar., 1942	Aug. 1, 1942
Basic Economic Considerations Affecting Single and Multiple Purpose Projects. *Otis M. Page*	Mar., 1942	Aug. 1, 1942
Early Contributions to Mississippi River Hydrology. *C. S. Jarvis*	Mar., 1942	Aug. 1, 1942
Stress Concentrations in Plates Loaded Over Small Areas. *H. M. Westergaard*	Apr., 1942	Sept. 1, 1942
Profile Curves for Open-Channel Flow. *Dwight F. Gunder*	Apr., 1942	
Discussion	June, 1942	Sept. 1, 1942
Model Tests on Structures for Hydroelectric Developments. *L. M. Davis*	Apr., 1942	
Discussion	Apr., June, 1942	Sept. 1, 1942
Design of Structural Members: First Progress Report of the Committee of the Structural Division on Design of Structural Members	Apr., 1942	Sept. 1, 1942
Classification of Irrigable Lands. *W. W. Johnston*	May, 1942	
Discussion	May, 1942	Oct. 1, 1942
Moment Balance: A Self-Checking Analysis of Rigidly Jointed Frames. *R. J. Cornish*	May, 1942	
Discussion	May, June, 1942	Oct. 1, 1942
Numerical Procedure for Computing Deflections, Moments, and Buckling Loads. *N. M. Newmark*	May, 1942	
Discussion	May, June, 1942	Oct. 1, 1942
Mechanics of Creep for Structural Analysis. *Joseph Marin*	May, 1942	Oct. 1, 1942
Relative Angular, Linear, and Traverse Accuracies in City Surveys. *S. A. Bauer*	May, 1942	Oct. 1, 1942
Physical Properties of Driven and Undriven Rivets of High-Strength Structural Steels: Progress Report of the Committee of the Structural Division on Structural Alloys	May, 1942	Oct. 1, 1942

NOTE.—The closing dates herein published are final except when names of prospective discussers are registered for special extension of time.

* Publication of closing discussion pending.

CONTENTS FOR JUNE, 1942

PAPERS

PAGE

Earth Pressure and Shearing Resistance of Plastic Clay: A Symposium 857

REPORTS

Protective and Remedial Measures for Sanitary and Public Health Engineering Services: Second Progress Report of the Sanitary and Public Health Engineering Division of the National Committee of the Society, on Civilian Protection in War Time 951

Advancement of Sanitary Engineering: Report of the Committee of the Sanitary Engineering Division on Advancement of Sanitary Engineering 965

DISCUSSIONS

Fundamental Aspects of the Depreciation Problem: A Symposium.
By Messrs. John F. Miller, and Fred Asa Barnes 997

Allocation of the Tennessee Valley Authority Projects.
By Messrs. Sherman M. Woodward, and Albert R. Arledge 1001

Drainage of Leveed Areas in Mountainous Valleys.
By W. B. Langbein, Assoc. M. Am. Soc. C. E. 1009

Protective and Remedial Measures for Sanitary and Public Health Engineering Services: Progress Report of the Sanitary and Public Health Engineering Division of the National Committee of the Society, on Civilian Protection in War Time.
By Messrs. James H. Le Van, Clarke Gardner, Michael J. Blew, and Gilbert H. Dunstan 1013

Hydraulic Design of Drop Structures for Gully Control.
By Walter T. Wilson, Assoc. M. Am. Soc. C. E. 1021

Analytical and Experimental Methods in Engineering Seismology.
By Messrs. George W. Housner, and Homer M. Hadley 1023

CONTENTS FOR JUNE, 1942 (*Continued*)

PAGE

Technique of Determining Shearing Strength of Soils: Progress Report of a Special Committee of the Soil Mechanics and Foundations Division on the Technique of Soil Tests.
By Jacob Feld, M. Am. Soc. C. E. 1031

Viscosity and Surface Tension Effects on V-Notch Weir Coefficients.
By A. A. Kalinske, Assoc. M. Am. Soc. C. E. 1033

Profile Curves for Open-Channel Flow.
By Carl Rohwer, Assoc. M. Am. Soc. C. E. 1035

Numerical Procedure for Computing Deflections, Moments, and Buckling Loads.
By M. S. Ketchum, Jr., Assoc. M. Am. Soc. C. E. 1037

Model Tests on Structures for Hydroelectric Developments.
By Adolf A. Meyer, M. Am. Soc. C. E. 1039

Timber Friction Pile Foundations.
By John W. Solomon, Esq. 1041

Design of St. Georges Tied Arch Span.
By Carlos A. Bejarano, Jun. Am. Soc. C. E. 1054

Energy Loss at the Base of a Free Overfall.
By Messrs. L. Standish Hall, and Hunter Rouse 1057

Moment Balance: A Self-Checking Analysis of Rigidly Jointed Frames.
By Messrs. A. A. Eremin, L. E. Grinter, and B. J. Aleck 1067

The Grease Problem in Sewage Treatment.
By W. S. Mahlie, Esq. 1071

Design of Sign Letter Sizes.
By Charles M. Noble, M. Am. Soc. C. E. 1073

For Index to all Papers, the discussion of which is current in PROCEEDINGS, *see page 2*

The Society is not responsible for any statement made or opinion expressed in its publications

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

EARTH PRESSURE AND SHEARING RESISTANCE OF PLASTIC CLAY

A SYMPOSIUM

PAGE

FOREWORD 858

Liner-Plate Tunnels on the Chicago (Ill.) Subway.
BY KARL TERZAGHI, M. AM. SOC. C. E. 862

Earth Pressure Measurements in Open Cuts, Chicago (Ill.) Subway.
BY RALPH B. PECK, JUN. AM. SOC. C. E. 900

Earth Pressure on Tunnels.
BY W. S. HOUSEL, M. AM. SOC. C. E. 929

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **November 1, 1942.**

FOREWORD

At the meeting of the Soil Mechanics and Foundations Division of the Society in Chicago, Ill., in October, 1941, three papers presented as different parts of the program contained interesting material on the subject of shear in plastic clays. Subsequently, it was decided to assemble these papers into a composite group designed for widespread discussion by the entire membership. The present Symposium is the result.

Valuable experience on the Chicago subway has been recorded by Karl Terzaghi, M. Am. Soc. C. E., and Ralph B. Peck, Jun. Am. Soc. C. E. The first paper concerns tunnel sections and the second open-cut sections.

Work described in these two papers was done by the Department of Subways and Superhighways, City of Chicago, under Edward J. Kelly, M. Am. Soc. C. E., Mayor, and Philip Harrington, M. Am. Soc. C. E., commissioner of the Department. Ralph H. Burke, M. Am. Soc. C. E., was chief subway engineer during the major part of the construction program, until Charles E. De Leuw, M. Am. Soc. C. E., became acting chief subway engineer. Joshua d'Esposito, M. Am. Soc. C. E., was project engineer for the U. S. Public Works Administration. Soil mechanics activities were administered by R. S. Knapp, who was in administrative charge of the Survey Section.

Professor Terzaghi was consulting engineer in matters pertaining to soil mechanics and Mr. Peck was in direct charge of soil investigations and the field observations. The parts of these papers dealing with the theory of earth pressure, with the interpretation of field data, and with the application of the findings to the design of the bracing in open cuts (for example, parts II to VI of the Peck paper) are based on memoranda prepared by Professor Terzaghi.

Contractors engaged in the work contributed materially to the success of the studies. On the Chicago subway these were the Healy Subway Construction Corporation, M. J. Boyle and Company, John C. Tully Company, Kenny Construction Company, M. Pontarelli and Sons, John March and Company, Robert R. Anderson Company, and Paschen Contractors, Inc. In addition, equipment for some of the tests on the Chicago subway was made available by the firm of Spencer, White and Prentis, Inc., through the kindness of Lazarus White, M. Am. Soc. C. E.

The third paper in the Symposium, by W. S. Housel, M. Am. Soc. C. E., contains detailed directions for determining the distribution of earth pressures on tunnels, which is part of the general research program in soil mechanics at the University of Michigan in Ann Arbor. This program consists of a more or less continuous series of investigations coordinated with construction projects and sponsored by the outside engineering organizations directly interested. The results reported by Professor Housel are derived from a project undertaken by the Department of Water Supply, City of Detroit, Mich. Laurence G.

Lenhardt, M. Am. Soc. C. E., general manager and chief engineer of that Department, was one of the originators of a soil mechanics research program in the Detroit area and has since been one of its most consistent supporters.

During the past several years the Armco Drainage Products Association has become actively associated in this work through their interest in earth pressure in underground excavations. A study of the accumulated earth-pressure measurements has been made and some related investigation undertaken and completed under the sponsorship of the Association. George Shafer, Assoc. M. Am. Soc. C. E., chief engineer, has been directly concerned in that development.

Notation.—The following letter symbols, adopted for standard use in the Symposium and subsequent discussion, conform essentially to American Standard Letter Symbols for Mechanics, Structural Engineering, and Testing Materials,[1] prepared by a Committee of the American Standards Association, with Society representation, and approved by the Association in 1932. In particular, they follow, as a guide, the Society's *Manual of Engineering Practice No. 22*, entitled "Soil Mechanics Nomenclature." Some restrictive definitions as modified by subscripts are given in the Symposium where they first appear.

A = area;
B = one half the width of a tunnel;
C = overload ratio; $S_1/S_c = C_1$; $S/S_A = C_2$;
c = cohesion of clay: c_s = cohesion of a substitute clay layer;
D = diameter of a tunnel section;
d = depth: d_1, d_2, etc., are the vertical distances between the ground surface and struts 1, 2, etc.;
F = resultant force normal to a curved sliding surface;
H = height and depth, as depth to the center of tunnel:
- H_b = depth to bottom of tunnel;
- H_c = depth of a cohesionless layer;
- H_t = depth to top of tunnel;
- H_z = depth below the surface to any point;
- H_1 = height of the tunnel arch springing above the invert; also H_1, H_2, H_3, etc. = depth from ground surface to struts 1, 2, 3, etc.;
- H_2 = distance from lowest strut to bottom of excavation;

K = pressure ratio, or coefficient of earth pressure at rest:
- K_A = pressure ratio for active earth pressure;
- K_s = hydrostatic pressure ratio for cohesionless material;
- K_0 = ratio between the horizontal and the vertical unit pressure at a given point in a sedimentary deposit;

L = length; length of a curved sliding surface: L_p = horizontal distance between vertical profiles of struts;
l = lever arm of a force with respect to its center of rotation:
- l_c = lever arm of force S at the ground surface;
- l_r = lever arm of the force P_A;
- l_w = lever arm of weight W;

[1] ASA-Z10a—1932.

N = center of pressure = ratio of the height of the center of gravity of the measured strut loads above the bottom of the cut, to the total depth of the cut;

n = coefficient of progressive failure;

P = a force:

- P_A = total lateral pressure from the surface to a depth H_z;
- P_b = a reaction;
- P_F = the magnitude of a resultant earth pressure analogous to a fluid of equal weight;
- P_l = lateral pressure of a fluid having a density equal to γ_a;

p = unit pressure intensity; also unit static pressure intensity:

- p_a = air pressure in excess of atmospheric pressure;
- p_h = horizontal pressure;
- p_N = maximum normal pressure;
- p_v = top vertical pressure;
- p'_2 = lateral pressure;
- p_3 = bottom vertical pressure under temporary equilibrium;

Q = total measured lateral pressure in one profile of bracing; also, total vertical load supported by sheeting: Q_1, Q_2, etc. = individual measured strut loads;

q = unconfined compressive unit strength or bearing capacity:

- q_a = average strength for depth H;
- q_c = compressive strength of clay;
- q'_c = strength of relatively undisturbed clay samples as determined by laboratory tests;
- q_D = bearing capacity of footings;
- q_m = minimum normal strength;
- q_r = strength of clay within given block;
- q_s = strength of a fictitious clay layer assumed to replace a sand layer;

r = radius:

- r_i = inside radius of a thick-walled cylinder;
- r_o = outer radius of a thick-walled cylinder;

S = effective shearing resistance of clay along one of the potential surfaces of sliding:

- S_A = average shear;
- S_c = cumulative average shear from the ground surface to the top of the tunnel;
- S_s = shearing resistance of a sand layer;
- S_v = average vertical shear;

t = time;

W = total weight; one half the weight of clay (solids and water combined) within a given shaded area, per foot length of tunnel; also weight of a wedge of soil mass within a failure zone:

- W_s = total weight of earth in a corner fillet;
- W_t = total weight of a tunnel;

w = water content:
- w_L = liquid limit;
- w_p = plastic limit;

γ = density; also unit weight of soil:
- γ_a = average density at a depth H;
- γ_c = average density of clay layer;
- γ_1 = density of sand deposit;

Δ = clearance allowed for jack, in making stress measurements;
ϵ = strain;
θ = angle of inclination from the horizontal;
σ = stress intensity:
- σ_A = active earth pressure;
- σ_h = horizontal stress on a vertical section;
- σ_v = vertical stress on a horizontal section;

ϕ = angle of internal shearing resistance of clay: ϕ_1 = angle of shearing resistance in clay.

—ED.

LINER-PLATE TUNNELS ON THE CHICAGO (ILL.) SUBWAY

BY KARL TERZAGHI,[2] M. AM. SOC. C. E.

SYNOPSIS

The initial subway system designed for Chicago, Ill., includes two routes, one called the State Street route and the other known as the Dearborn Street route. The locations of these two routes with reference to the lake shore and the Loop District are shown in Fig. 1. Each route consists of station sections

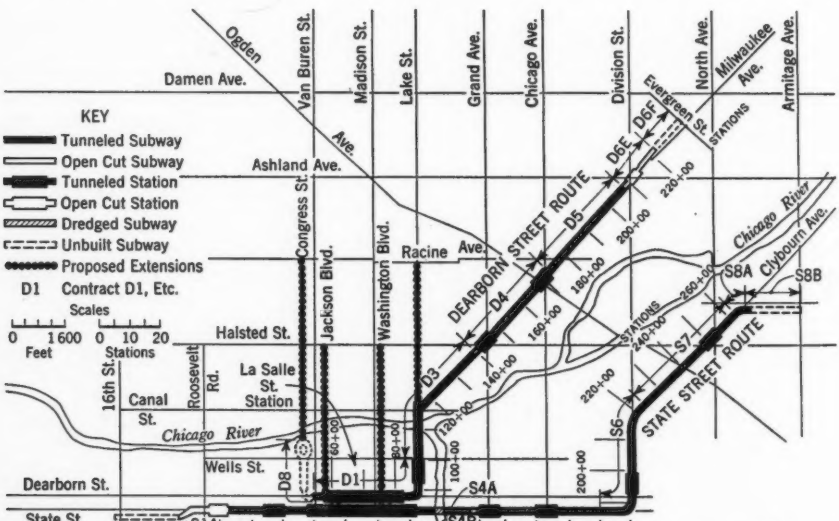


FIG. 1.—INITIAL SUBWAY SYSTEM; CHICAGO, ILL.

which are connected with each other by a double tube. Each tube has a clear width of 15 ft and a clear height of 15 ft. The bottom of the tubes is approximately 50 ft below the street surface. Between about 15 ft and 60 ft below the street surface, the soil consists of blue clay, the consistency of which ranges between stiff and very soft. The system comprises a total of 7.7 miles of double tubes and station sections. Of this total, 5.7 miles have been constructed by the light liner-plate method and 2.0 miles by the use of shields. In both types

[2] Winchester, Mass.

of tunnels an air pressure of about 12 lb per sq in. in excess of atmospheric pressure was maintained.

The streets, under which the subway is located, carry street-car and elevated lines and are used by heavy trucks. Buildings from one to eighteen stories border the streets from one end of the subway routes to the other. The foundations of the buildings vary from spread footings 4 ft to 6 ft below the ground surface to caissons founded on bedrock. To serve these buildings utilities of every description are located between the street surface and the roof of the tubes.

Before construction was started, the following questions had to be answered:

(*a*) What soil investigations should be made in order to obtain a general idea of the soil conditions along the proposed routes?

(*b*) Which sections can be constructed by means of the light liner-plate method and where will it be necessary to use the more expensive shield method?

(*c*) What are the intensity and the distribution of the clay pressure on the permanent tubes?

During construction two more questions arose—namely,

(*d*) What should be done to reduce the settlement of the street surface to a minimum? and

(*e*) How can catastrophic consequences of mining into a pocket of unexpectedly bad material be avoided?

When attempting to answer such questions, no theory and no method based on soil tests can be relied upon, unless the validity of the conclusions has previously been demonstrated by reliable measurements in the field. In connection with soft clay tunneling no such measurements had ever been made. Therefore the Subway Department decided to fill the gaps in current knowledge by systematic observations supplemented by soil investigations and to apply the results to the problems of construction as experience increased. As a by-product, the Department accumulated a wealth of information which can be turned into profit in connection with the proposed extensions to the initial system.

The paper contains an abstract of the findings pertaining to the liner-plate tunnels. The first part deals with the conditions for the equilibrium of the liner-plate support, with various theoretical conceptions concerning the clay pressure on the finished tunnel tubes and with the causes of the settlement of the street surface. The second part contains a brief review of the soil investigations. In the third part the reasons for local application of the shield method are discussed. The fourth part deals with the results of observations, made in

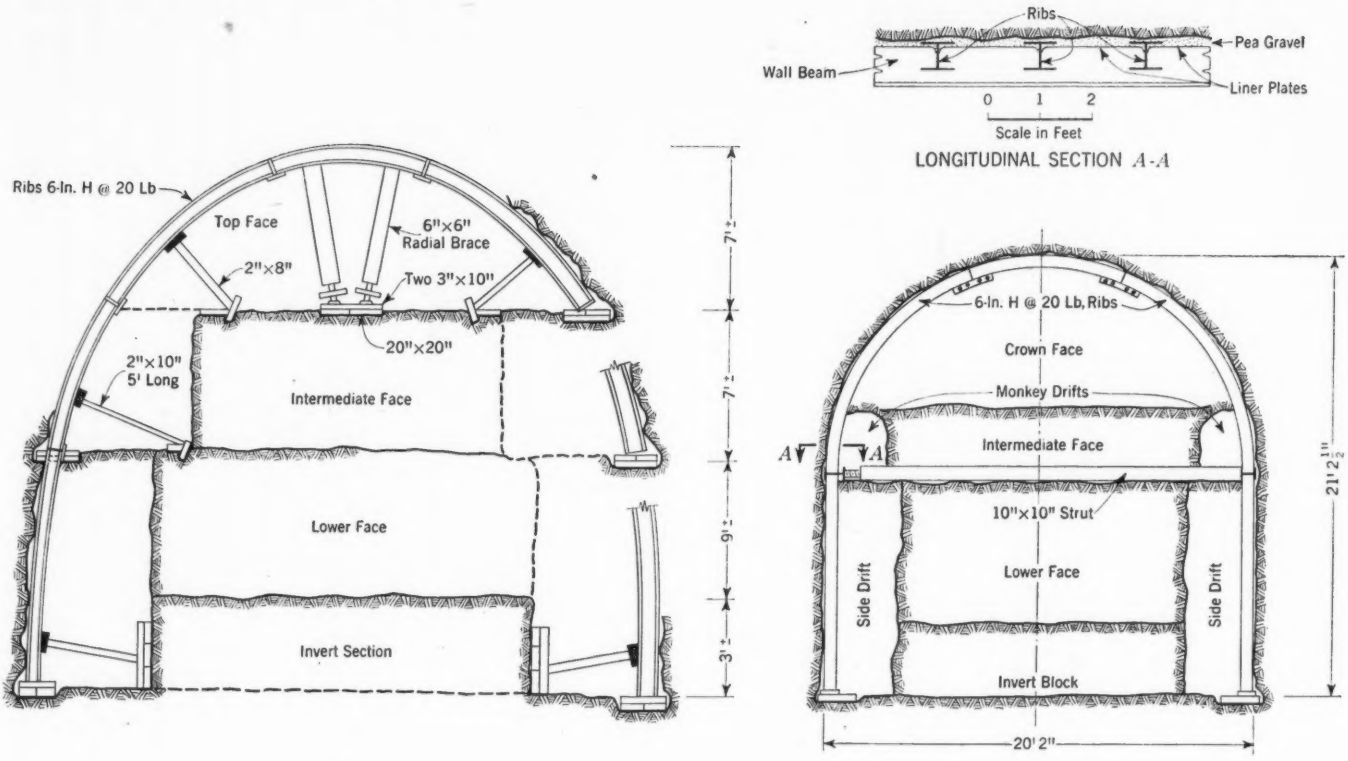


FIG. 2.—CHARACTERISTIC TUNNELING OPERATIONS

an experimental section, concerning the clay pressure on the finished tubes, and the fifth concerns the settlement of the street surface. Another paper concerned with the shield tunnels is appearing elsewhere.[2a]

I.—Clay Pressure on Temporary and on Permanent Tunnel Support

Liner-Plate Method of Tunneling in the Chicago Subway.—Fig. 2 illustrates the two principal liner-plate methods used on the Chicago subway. The ribs consist of 6-in., 20-lb I-beams spaced 2 ft to 2.5 ft on centers. The space between the ribs is bridged by the liner-plates. The individual sections of the liner-plates were either very long and were bolted to the web of the ribs, or they were narrow (lagging plates) and freely supported on the outside flanges of the ribs. Since it is not possible to make the excavation exactly fit the liner-plates, they were inevitably separated from the clay by an empty space varying from zero to several inches. Three different methods were used for filling this space: Pumping a sand-cement grout through holes in the liner-plates; blowing pea gravel through holes in the liner-plates by means of compressed air; or driving wooden wedges into the open space between the clay and the lagging plates.

According to the method illustrated by Fig. 2(*a*) each rib was installed in sections starting at the crown. In every stage of construction the two lower ends of the arch ribs rested on footings except during the brief periods when new sections are added. Mining started at the crown (top face) and ended with the excavation of the invert section.

When the method illustrated by Fig. 2(*b*) was used, the attack was made by first excavating two unlined monkey drifts. The bottom of each monkey drift served as a support for a wall beam with a length of 6 ft to 16 ft. The length of the monkey drift ranged between one and one and one-half times the length of the wall beam, which in turn transferred the load on to the footings of the vertical sections of the ribs.

In both types of tunnels, the size of the footings ranged from 0.8 and 2 sq ft per lin ft of the tunnel. The small footings consisted of square blocks and the large ones were continuous. To strengthen the lateral support of the clay, horizontal struts were inserted between the ribs below midheight of the tunnel or between the wall beams as shown in Fig. 2(*b*). Customarily, two or three shifts excavated the working face for a distance of 25 to 40 ft. The invert was then cleared, reinforcing steel laid, and the concrete invert poured. In the next step the concrete arch was concreted, after which sand-cement grout was injected through pipes in the wall to fill the voids between the concrete and the liner-plates and the remaining open spaces between the liner-plates and the clay.

Conditions for the Stability of the Liner-Plate System.—Figs. 3(*a*) and 3(*b*) show simplified sections through the temporary support of the clay in a Chicago liner-plate tunnel. In both figures the shaded area represents the block of clay that moves downward if the support fails. If, by chance, the lower part of the tunnel is located within an exceptionally stiff layer of clay between soft

[2a] *Journal*, Boston Soc. of Civ. Engrs., July, 1942.

layers, the potential surfaces of sliding consist of vertical planes through the edges of the bottom of the tunnel and the failure occurs approximately as shown in Fig. 3(*a*). Let

W = one half of the weight of the clay (solid and water combined) located within the shaded area, per foot of length of the tunnel;
A = one half of the area of the footings, per foot of length;
B = one half the width of the tunnel;
q_D = bearing capacity of the footings per square foot;
S = the effective shearing resistance of the clay along one of the potential surfaces of sliding, per foot of length of the tunnel; and
p_a = the air pressure in pounds per square foot, in excess of the atmospheric pressure.

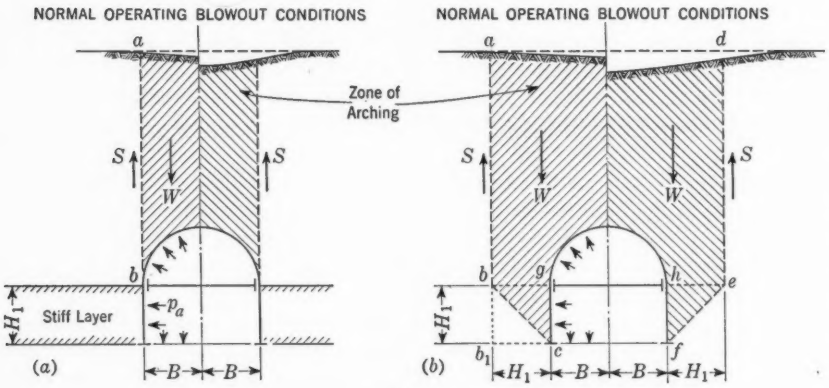


FIG. 3

For reasons stated in Section II on soil investigations, it will be assumed in the following estimate of the bearing capacity of the support of the clay that the angle of shearing resistance ϕ of the clay is equal to zero. If $\phi = 0$, a uniformly distributed pressure p_a per unit of area on the bottom of the tunnel increases the bearing capacity of the footings from q_D to $q_D + p_a$ per unit of area, regardless of whether p_a is exerted by a surcharge or by compressed air.[3] The forces that act on the clay adjoining the tunnel under normal working conditions are shown on the left-hand side of Fig. 3(*a*). The temporary support of the roof fails unless

$$W - S = p_a B + A\,(q_D + p_a) \quad \ldots\ldots\ldots\ldots\ldots\ldots (1a)$$

After a complete blowout $p_a = 0$, whereupon the stability of the support requires that

$$W - S = A\,q_D \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1b)$$

If the clay on both sides of the tunnel is not exceptionally stiff, the width of the block of clay that sinks down in case of a failure has approximately a

[3] "Erdbaumechanik auf bodenphysikalischer Grundlage," by Karl Terzaghi, Leipzig, Deuticke, 1925.

width of $2\,(B + H_1)$, as indicated in Fig. 3(*b*). For the purpose of a rough evaluation of the stability conditions, the potential surfaces of sliding can be replaced by two sets of plane surfaces represented by the lines a b c and d e f, respectively. The prism of clay located below the horizontal section b g is acted upon by a vertical load $W - S - p_a B$. Part of this load is carried by the prismatic block b b_1 c g and the remainder by the footings of the ribs. The average unconfined compressive strength q_r of the clay located within the block is known from tests on cylindrical samples with a height of 3.5 in. and a diameter of $1\frac{7}{8}$ in. The height of the block is equal to its width, H_1. No lateral displacement can occur along the surface b g and the tendency to bulge toward the tunnel is resisted by the stiffness of the adjoining ribs and by the adhesion between the base of the footings and the clay. If necessary, the resistance against lateral yield of the lower ends of the ribs can be increased by horizontal braces. The flexural strength of the ribs combined with the resistance of the footings against lateral displacement increases the unit load required for producing a failure of the block from q_r to at least $2\,q_r$. A horizontal unit pressure p_a on the vertical side of the block further increases this value from $2\,q_r$ to $2\,q_r + p_a$. The bearing capacity of the footings is $A\,(q_D + p_a)$. As a condition for the adequate support of the roof of the tunnel:

$$W - S - B\,p_a = (2\,q_r + p_a)\,H_1 + A\,(q_D + p_a) \ldots\ldots\ldots (2a)$$

A complete blowout reduces p_a to zero, whereupon

$$W - S = 2\,q_r\,H_1 + A\,q_D \ldots\ldots\ldots\ldots\ldots\ldots (2b)$$

In either case the failure would be associated with a bending and buckling of the vertical legs of the ribs, preceded by a sinking of the footings. Other types of failure are conceivable, but they are not likely to occur because almost everywhere on the Chicago subway the stiffness of the clay increases quite rapidly with increasing depth below the bottom of the tunnels.

Eqs. 1 and 2 represent the results of a crude estimate. However, more elaborate computations would be entirely out of proportion with the approximations involved in the estimate of the shearing resistance of irregularly stratified masses of clay that have been explored by means of drill holes spaced at 300 ft.

Clay Pressure on the Finished Tubes.—In the vicinity of the working face the clay is in a state of slow, viscous flow toward the tunnel. After a few days this movement becomes imperceptible. However, if the mining operations were discontinued, the movement would undoubtedly proceed at a very low rate, and the support would adjust itself to the movement of the clay. This is a process of slow deformation produced by a constant state of stress in the clay. The construction of the concrete tube introduces a practically rigid obstacle within the clay. If the moving clay meets a rigid obstacle, the pressure of the clay on the obstacle gradually increases, and the major part of the load, which was previously carried by arching, is transferred to the obstacle. This conclusion is in accordance with the observations on the Chicago tunnels, and it precludes the application of the reasoning illustrated by Fig. 3 and expressed by Eqs. 1 and 2 to the computation of the clay pressure on the reinforced

concrete tubes. Still less applicable is Rankine's earth-pressure theory because it is based on the assumptions that the clay is everywhere in a state of plastic equilibrium and that the mining operations do not change the state of stress in the clay. All these facts are known to most engineers who have had experience in tunnel construction; yet no other theories are available. Hence, in the past, tunnel tubes have been designed on the basis of empirical rules whose deficiencies were compensated by a wide margin of safety. Experience acquired in the course of the construction of the Chicago sewer system led to the tentative conclusions that the roof of the tunnels carries the full weight of the overburden and that the ratio between the horizontal and the vertical normal stress at a given point of the clay may have any value between 1/3 and 2/3. The sewer tunnels which have been designed on the basis of these assumptions have stood up very well and, since most of them are rather small, the thickness of the walls does not appear to be excessive. However, when applying the same rule to the design of large tubes such as those of the Chicago subway, one finds that the tunnels require very thick walls provided with heavy reinforcement and one faces the question of whether the computed dimensions may not be excessive.

It would be idle to attempt to answer this question on purely theoretical grounds, because the final state of stress in the tubes depends to a large extent on the state of stress in the clay after mining, which in turn depends not only on the method of mining but also on the workmanship. However, the mechanical properties of the soft clay, as disclosed by observations in the tunnel, suggest the following possibility, illustrated by Fig. 4. This figure shows a simplified section through a tunnel tube designed on the assumption that the ratio between the horizontal and the vertical pressure in the clay is everywhere equal to an arbitrarily selected constant—for instance 7/8. If the real value of this ratio is smaller (for instance, 1/2) a slight increase of the width of the tunnel is sufficient to increase the pressure ratio in the clay adjoining the tunnel from 1/2 to 7/8. A small change of the horizontal diameter does not injure the tunnel tube unless the walls are very thick. Hence, it is likely that a thin concrete shell with a nominal reinforcement should suffice permanently for withstanding the clay pressure. On account of the economic importance of this possibility, the Subway Department authorized the construction of two experimental sections for investigating the deformations associated with the stress adjustment in the clay adjoining a flexible tube. One of the sections is located in a liner-plate tunnel, on contract S-6 (Fig. 1) and the other one in a shield tunnel, on contract S-3. The results of the observations on the liner-plate section are described, subsequently, in Section IV.

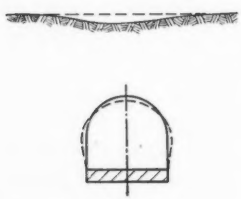


FIG. 4

Settlement of the Street Surface Above Liner-Plate Roof Supports.—During the process of mining, the arch ribs and the compressed air carry only the difference between the weight of the clay located above the excavation and the shearing forces which oppose the downward movement of the clay (see Fig. 3). This was demonstrated, for instance, by strain-gage measurements in the legs

of the arch ribs on contract D-3. The weight transmitted by the arch ribs on to the footings represented the equivalent of an overburden of 5 ft; the compressed air carried the equivalent of 15 ft; and the total depth of the overburden was 55 ft. As a matter of fact, without effective cooperation of arching above the tunnel, the liner-plate method of construction would not be conceivable. For mechanical reasons arching is inevitably associated with a trough-like depression of the surface of the ground located above the tunnel. Since the volume of the clay remains practically unchanged, the ground surface cannot settle unless the clay advances simultaneously toward the excavation. The movement can be resolved into three components, a downward movement (crown settlement), a horizontal yield toward the working face (axial squeeze), and an inward movement of the sides of the tunnel (lateral squeeze). On account of the mechanical properties of undisturbed clays a very small yield suffices to develop all the strength the clay has. Any further yield of the clay either has no influence on the pressure on the support, or it increases the pressure as it does in open cuts. Hence a certain part of the settlement of the ground surface is inevitable and useful. Everything in excess of this part is harmful and should be avoided.

If the support of the roof of the tunnel is adequate, the rate of the yield of the clay toward the excavation should decrease rapidly like the rate of settlement of an adequately dimensioned footing on clay. On the other hand, if the load on the liner-plate support is excessive, the roof deflects downward at a constant or an accelerated rate until failure occurs. Therefore the rate of subsidence of the roof can be utilized as a means of determining whether or not the support is loaded to the point of failure. The methods of observation and interpretation are discussed in Section V.

II.—Soil Investigations

Geological Conditions.—Between the ground surface at an average elevation of about 12 ft above the level of Lake Michigan and at a depth of roughly 100 ft below the surface, the subsoil of Chicago consists of an irregularly stratified glacial deposit whose surface is covered with a layer of silt, silty sand, and artificial fill with a thickness of from 10 to 15 ft. The bedrock consists of Niagaran Limestone. Within the glacial deposit the following principal strata can be distinguished:

Stratum	Description
a	A top crust of hard, yellowish or bluish clay with a thickness of 3 to 10 ft.
b	An irregularly stratified bed of soft blue clay whose stiffness increases in a general way with increasing depth below the top crust. The thickness of this bed ranges between 60 ft and 80 ft. The bed contains a variable percentage of small rock fragments (sand to pea-gravel size, rarely greater) and isolated, more or less distorted pockets and lenses of silt and sand.
c	A layer of hard, gray, sandy material, commonly called hardpan, which may or may not be separated from the overlying stratum b by a layer of very hard, bluish clay.

The bottom layer c seems to represent the remnants of a ground moraine. Its base is usually separated from the surface of the bedrock by a layer that consists of sand intermingled with silt, gravel, and boulders. In some places the base layer contains natural gas and appears to be very permeable. The soft clay b located above the hardpan is assumed to have originated by direct deposition out of the melting ice at a time when the lake level was higher than at present. The stiff top crust (a) was formed by desiccation and local oxidation of the soft blue clay during a temporary low stage of the lake which occurred some time after the ice had retreated permanently from the Chicago area. During the same stage flowing water eroded little valleys, here and there, into the soft blue clay. These valleys have subsequently been filled with sand and silty sand. Near the intersection of South State and 16th streets, the bottom of one of these gullies was found to be about 34 ft below the present lake level. It was separated from the underlying soft blue clay by a hard crust similar to the stratum a which is commonly encountered immediately above or below the present lake level.

Practically all the tunnels are located within the soft blue clay (stratum b). The hardpan layer c was not encountered except at the river crossings.

Type and Depth of Test Borings.—Since the stiffness of the bottom has a decisive influence on the bearing capacity of the temporary support of the roof, all the test borings were carried to a depth of about 10 ft below the bottom elevation, unless the bottom was located below the surface of the hardpan. The drilling equipment consisted of that commonly used for wash borings, with the additions required for securing continuous 2-in. seamless tube samples of the clay in 2.5-ft sections. The diameter of the casing was 2.5 in. The first operation consisted in breaking the pavement. The hole was made to a depth of about 6 ft with a post hole digger; and below this depth and down to the surface of the uppermost clay stratum (a), a washing bit was used. The samplers were forced into the clay by static load. As a supplement to the soil tests the static load required to force the sampler into the clay has been recorded. If the maximum load of about 540 lb did not suffice to produce penetration, the sampler was driven into the clay by a 125-lb weight used as a hammer (2.5 ft drop and about 40 blows per min). However, this procedure was only used in some of the holes for penetrating the hardest part of the top crust.

The bottom of all the tunnels except those at the river crossings is at a considerable elevation above the base of the soft clay stratum b. Therefore only one of the test borings was driven to bedrock (which was encountered at a depth of about 100 ft) and one of them was discontinued within the hardpan stratum, at a depth of about 80 ft. Both these borings are close to the La Salle Street station. All the others terminated within the lower and stiffer part of the soft blue clay.

Spacing of Drill Holes and Variation of Soil Properties Between the Drill Holes.—It was the writer's intention to make one drill hole every 200 ft. However, on account of the time limit set for the completion of the survey, this distance was increased to about 300 ft. The average depth of the drill holes was approximately 60 ft; the total length of the drill holes on the initial system

is 8,376 ft, and 3,548 ft have been added along the proposed extension lines. Fig. 5(*a*) shows an ideal section through the principal strata with two drill holes 300 ft apart. Considering the geological origin of the deposit and the resulting erratic structure, it appears hopeless to draw from the test-boring

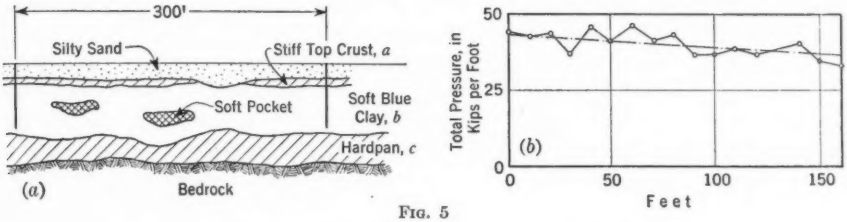


FIG. 5

records any conclusions concerning the soil conditions within the space between the holes. However, when dealing with irregularly stratified water-laid deposits, the writer has been repeatedly impressed by the fact that the average soil properties do not change, in a horizontal direction, nearly as erratically as the individual records would lead one to expect. The observations on the Chicago subway have revealed the same fact in connection with the ice-laid deposit of this region. This is illustrated by Figs. 5(*b*) and 6.

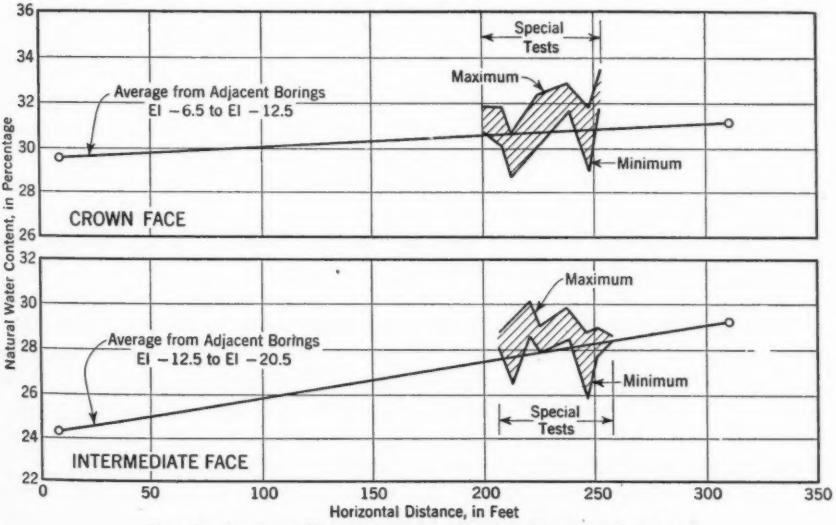


FIG. 6.—AVERAGE WATER CONTENT, TUNNEL HEADING SAMPLES

Fig. 5(*b*) represents the variation of the lateral pressure on the timbering in an open cut with a length of 160 ft and a depth of about 36 ft, on contract S-1A (south end of South State Street, Fig. 1). The ordinates represent the measured lateral pressure in kips per linear foot of the cut. They are indicative of the average shearing resistance of the soil on both sides of the cut. The individual values do not deviate by more than $\pm 16\%$ from those represented by the straight, broken line.

Fig. 6 shows the variations of the natural water content of the clay over a length of 50 ft of the crown face and the intermediate face of the tunnel on contract S–5 (Fig. 1) between Station 144+59 and Station 155+03. The dotted vertical lines in Fig. 7 represent the lines along which the samples were taken at 6-in. vertical intervals. A set of such samples was obtained for approximately every 5-ft advance in the working faces until eight complete sets were taken. Each of the 24 vertical rows obtained was considered a potential boring. In order that the importance of the variations of the water content in horizontal directions could be visualized, the average water content in the crown face and in the intermediate face was computed for each potential boring represented by a dotted line in Fig. 7. The maximum and minimum values of these averages for each face at a given position of the heading are plotted in Fig. 6. The ordinates of the straight line a b represent the values of the water content which were to be expected on the basis of interpolation between the values found for material from the regular test borings at each end of the section investigated. Fig. 6 shows that the greatest difference between the real values and those represented by the lines a b does not exceed ±7%, corresponding to a variation of the water content of about ±2%.

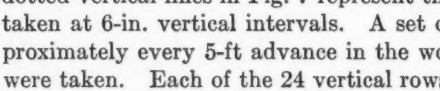


FIG. 7.—LOCATION OF TUNNEL HEADING SAMPLES (Spacing, 6 In. Vertically)

These data and the results of all the other variation surveys seem to justify the conclusion that every group of five or six text borings, driven along a section with a length of 1,200 or 1,500 ft, can be considered indicative for the general character of the entire section.

Soil Testing.—The testing program was governed by the practical purpose of the investigations. Observations were made: (*a*) To learn in advance whether the soil conditions on the different contracts were "good," medium, or "bad"; (*b*) to determine on which contracts it would be necessary to use a shield; and (*c*) to correlate the results of noteworthy observations and experiences with the properties of the clay adjoining the seat of the observations—otherwise the results would have been practically worthless.

The difficulties associated with the mining operations depend almost exclusively on the shearing resistance of the clay located above, on both sides, and immediately below, the bottom of the tunnel. During the ten years since 1932 the shearing resistance of clay has been thoroughly studied by many investigators. Important contributions were made in 1936 by L. Rendulič[4] and in 1939 by M. J. Hvorslev.[4,5] All students of the subject have demonstrated, for both undisturbed and remolded samples, that the shearing resistance of the clay in-

[4] "Relation Between Void Ratio and Effective Principal Stresses for a Remolded, Silty Clay," by L. Rendulič, *Proceedings*, International Conference on Soil Mechanics, Cambridge, Mass., 1936.

[5] "Torsion Shear Tests and Their Place in the Determination of the Shearing Resistance of Soils," by M. J. Hvorslev, *Proceedings*, A. S. T. M., Vol. 39, 1939, p. 999.

creases rapidly with the pressure under which the clay has been consolidated in the laboratory. It is very probable—although it has not yet been conclusively substantiated by field observations—that the disturbed clay in a rolled or puddled earth dam exhibits similar characteristics. However, during his entire professional career, the writer has not yet seen any evidence that aforementioned conclusions based on the test results also apply to soft clay deposits in a natural state. Furthermore, he has found repeatedly (in striking contrast to what one should expect from the test results) that slopes on soft, undisturbed clays fail if the average shearing stress on the potential surface of sliding becomes roughly equal to one half of the average nonconfined compressive strength of fairly undisturbed samples, regardless of the depth of the overburden.[6] On the other hand, no one has ever published any field data that would demonstrate adequately the existence of a pressure-conditioned shearing resistance in soft clay deposits in other localities. Theoretical arguments do not count in connection with this practical issue. Hence, before construction was started, the writer considered the nonconfined compressive strength of fairly undisturbed samples more indicative for the shearing resistance of the clay in the field than any other quantity that can be determined by laboratory tests.

Observations during construction merely strengthened the writer's original confidence in the compressive strength test. The digest of the pressure measurements in open cuts demonstrated for the Chicago clays the existence of a simple statistical relation between the lateral pressure and the average nonconfined compressive strength which appears to be analogous to the relation between shearing resistance and compressive strength for soft clay slides, which has been mentioned previously herein. This is the subject of the second paper in the Symposium, by R. B. Peck, Jun. Am. Soc. C. E. No evidence has been found that the angle of shearing resistance of the clay is appreciably in excess of zero degrees at any point. The discussion of the physical causes of this phenomenon is beyond the scope of this paper.

In order to make the routine soil tests, the Subway Department established a soils laboratory under the personal direction of Mr. Peck. It occupied an area of 30 ft by 20 ft and was equipped with a very satisfactory humid room. The personnel consisted of eight young assistants, most of whom worked part of the time in the laboratory and part of the time in the field (on cut measurements, observations in the experimental sections, etc.).

By loading a great number of nonconfined seamless tube samples ($3\frac{1}{2}$ in. long, $1\frac{7}{8}$ in. in diameter) to the point of failure within a time ranging between 2 and 20 min, it was found that, within this range, the time factor is immaterial. Therefore it was decided to run the tests within the shortest time compatible with satisfactory technique. This time was 5 min.

Other series of tests demonstrated that the average nonconfined compressive strength of seamless tube samples is equal to 75% of the compressive strength of samples that have been carefully cut with a knife out of the same clay in the tunnel. On the other hand, the pressure measurements in the open cuts led to the conclusion that progressive failure reduces the average compressive

[6] "Stability of Slopes of Natural Clay," by K. Terzaghi, *Proceedings*, International Conference on Soil Mechanics, Cambridge, Mass., Vol. I, pp. 161–165.

strength of the clay by about 25%. Since the angle of shearing resistance is practically equal to zero, the shearing resistance of the clay is approximately equal to one half of its compressive strength. This supports the conclusion that the shearing resistance against progressive shear failure in the field is approximately equal to one half of the average compressive strength of the seamless tube samples. Complete remolding reduced the nonconfined compressive strength of undisturbed tunnel samples to about one fourth of this value.

Although the time for determining the nonconfined compressive strength of one seamless tube sample does not exceed 5 min, the construction program did not permit making a compression test on every 6-in. section of every sample. However, by plotting a great number of nonconfined compressive-strength values against natural water content, it was found that a fairly consistent statistical relation exists between these two quantities for the soft Chicago clay. Therefore the following procedure was adopted: Every 2.5-ft seamless tube sample was cut into six sections. The natural water content was determined for each of the six sections, but the nonconfined compression test was made on only one of them.

The data thus obtained were used for plotting curves of equal compressive strength on vertical sections through the center line of the tunnel. Fig. 8(*a*) shows such a plot for the South State Street part of the State Street route and Fig. 8(*b*) for the North State Street and the Division Street part. The plots reveal at a glance the difference between the soil conditions south and north of the Chicago River. On South State Street, in the Loop District, the bottom of the tunnel is almost entirely located above the 0.8 kg per sq cm line, indicating relatively soft bottom and that more than 85% of the length of the roof is located within very soft material with a compressive strength of less than 0.4 kg per sq cm. On the other hand, along the northern part of the route, shown in Fig. 8(*a*), the bottom of the tunnel is almost consistently firm and the major part of the roof is located within medium stiff clay.

As a supplement to the aforementioned routine tests, Atterberg limit and triaxial compression tests were made. Since these tests had no direct bearing on the practical purpose of the investigations, they were made only on a limited number of samples. They furnished the following data, which may serve for comparing the Chicago clay with soft clay deposits in other localities. The plastic limit w_p of the uppermost and softest part of the soft clay deposit (designated as stratum b) was about 19% and that of the lower, stiffer part 18%. The corresponding values of the liquid limit, w_L, were 40% (exceptionally up to 50%) for the uppermost and about 30% for the lower part. For the hardpan on contract D-3 the values $w_L = 18\%$ and $w_p = 14\%$, indicating a nonplastic material, and for the hard blue clay at the upper boundary of the hardpan the values $w_L = 21\%$ to 25% and $w_p = 16\%$ have been obtained.

Triaxial compression tests at constant water content after complete consolidation under different hydrostatic pressures up to 4 atmospheres have been made on two sets of samples. One set was for the stiffer clays in the lower part of the stratum b and the other one for the softest material which is occasionally encountered in the uppermost part of this stratum. The tests on the

first series were made by P. C. Rutledge at Purdue University in Lafayette, Ind., and those on the second series by D. W. Taylor, Assoc. M. Am. Soc. C. E., at Massachusetts Institute of Technology in Cambridge, Mass. The tests yielded the values 11° (softest clay) and 18° (stiffer varieties), respectively, for the angle of shearing resistance. These values are only of theoretical interest, because they have no relation to the shearing resistance of the clays in connection with cut and tunnel problems.

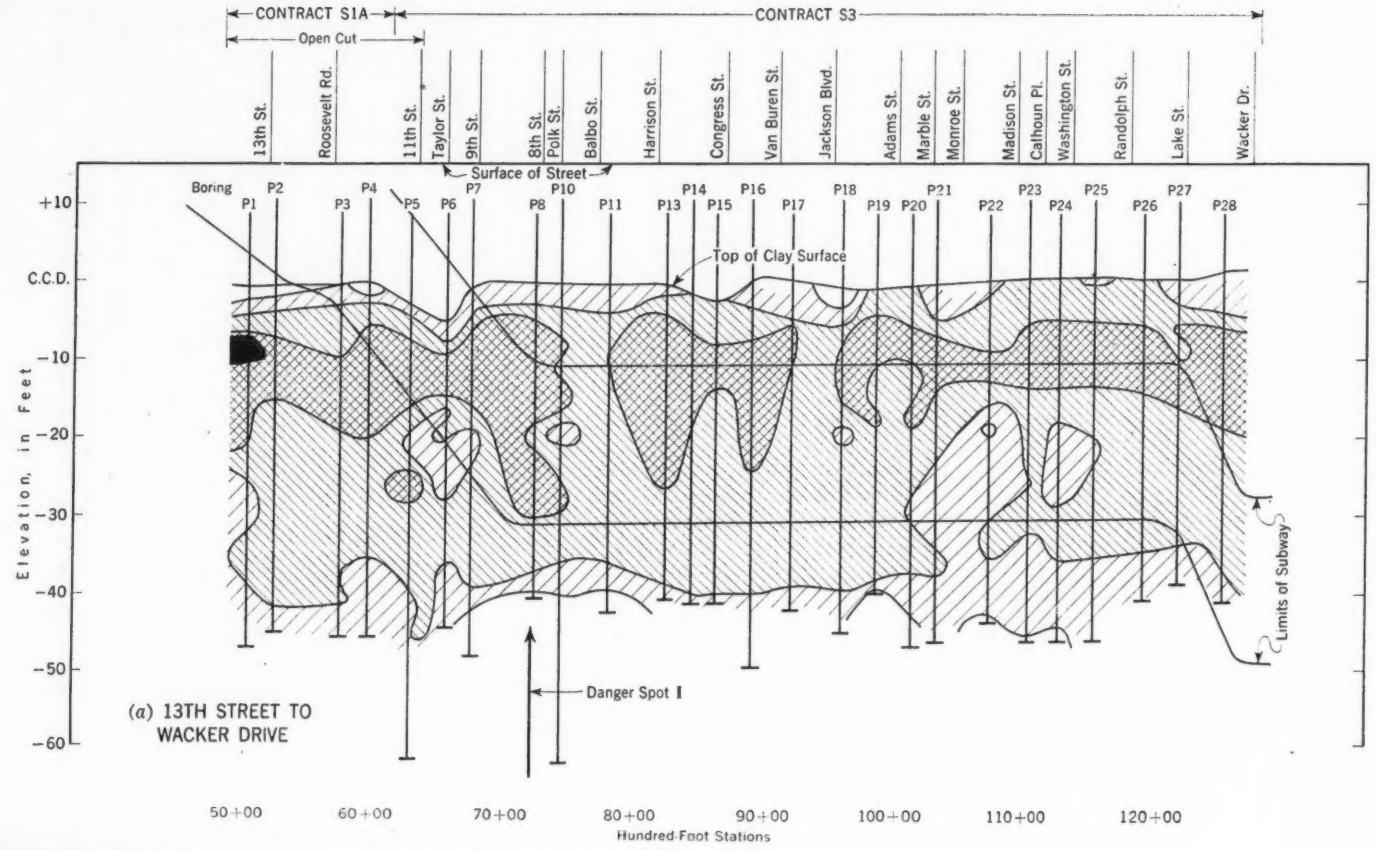
Cost and Degree of Efficiency of the Soil Investigations.—The cost of the borings was about $2 per ft of drill hole. The soil tests required for preparing the soil profiles included about 6,000 nonconfined compressive-strength tests and about 25,000 water-content determinations. The cost of boring, testing, and plotting amounted to about 42¢ per linear foot of single track of subway. Considering the importance of knowing in advance the general character of the soil conditions for the different contracts, these costs are insignificant.

Two years of experience with soil investigations on the subway led the writer to the following conclusions regarding the relative merits and shortcomings of the adopted procedure: In general, the type of boring, the spacing of the borings, and the program for soil testing were satisfactory. However, the stiff crust (stratum a) located above the soft blue clay deserves more attention than it received, because its shearing resistance has an important influence on the pressure on the temporary support in tunnels and open cuts. Along open cuts the spacing of the drill holes should be reduced to not more than 100 ft, and the nonconfined compressive strength should be determined on at least 2 or 3 samples per 2½-ft section, because knowledge of this subject has advanced to a point where the data can be used to considerable advantage in the design of the bracing system in the open cuts. (This phase is demonstrated in the paper by Mr. Peck in this Symposium.)

For most of the tunnel sections of the subway a drill-hole spacing of 300 ft is sufficient because the results inform the engineer whether the section is "good," medium, or "poor." If a section is "good" or medium, he is not interested in finding out how "good" it is. However, at the danger spots, where the clay is weakest (intersection between South State Street and 8th Street or intersection between North Clark Street and Division Street) intermediate borings should be made until one has a fairly clear conception of what the worst conditions are. The practical value of this knowledge is discussed in Section III.

III.—Liner-Plate Versus Shield Tunnel

General Conditions for the Safe Application of the Liner-Plate Method.—A total of 5.7 miles of the double tubes of the Chicago subway were constructed by the light liner-plate method and the remainder, with a total length of 2.0 miles, by the shield method. The contract price for the shield tunnels was approximately $580, and that for the liner-plate tunnels only $270, per linear foot of a single tube. However, the liner-plate method cannot be used safely unless the temporary support of the tunnel is capable of withstanding the clay pressure, at least for a short time, in the event of a complete blowout, and in very soft ground it cannot be used at all on account of inadequate support of


CONTRACT S1A
CONTRACT S3
Open Cut
13th St.
Roosevelt Rd.
11th St.
Taylor St.
9th St.
8th St.
Polk St.
Balbo St.
Harrison St.
Congress St.
Van Buren St.
Jackson Blvd.
Adams St.
Marble St.
Monroe St.
Madison St.
Calhoun Pl.
Washington St.
Randolph St.
Lake St.
Wacker Dr.
Surface of Street
Boring
P1 P2 P3 P4 P5 P6 P7 P8 P10 P11 P13 P14 P15 P16 P17 P18 P19 P20 P21 P22 P23 P24 P25 P26 P27 P28
Top of Clay Surface
Limits of Subway
Danger Spot I
+10
C.C.D.
-10
-20
-30
-40
-50
-60
Elevation, in Feet
(a) 13TH STREET TO WACKER DRIVE
50+00
60+00
70+00
80+00
90+00
100+00
110+00
120+00
Hundred-Foot Stations

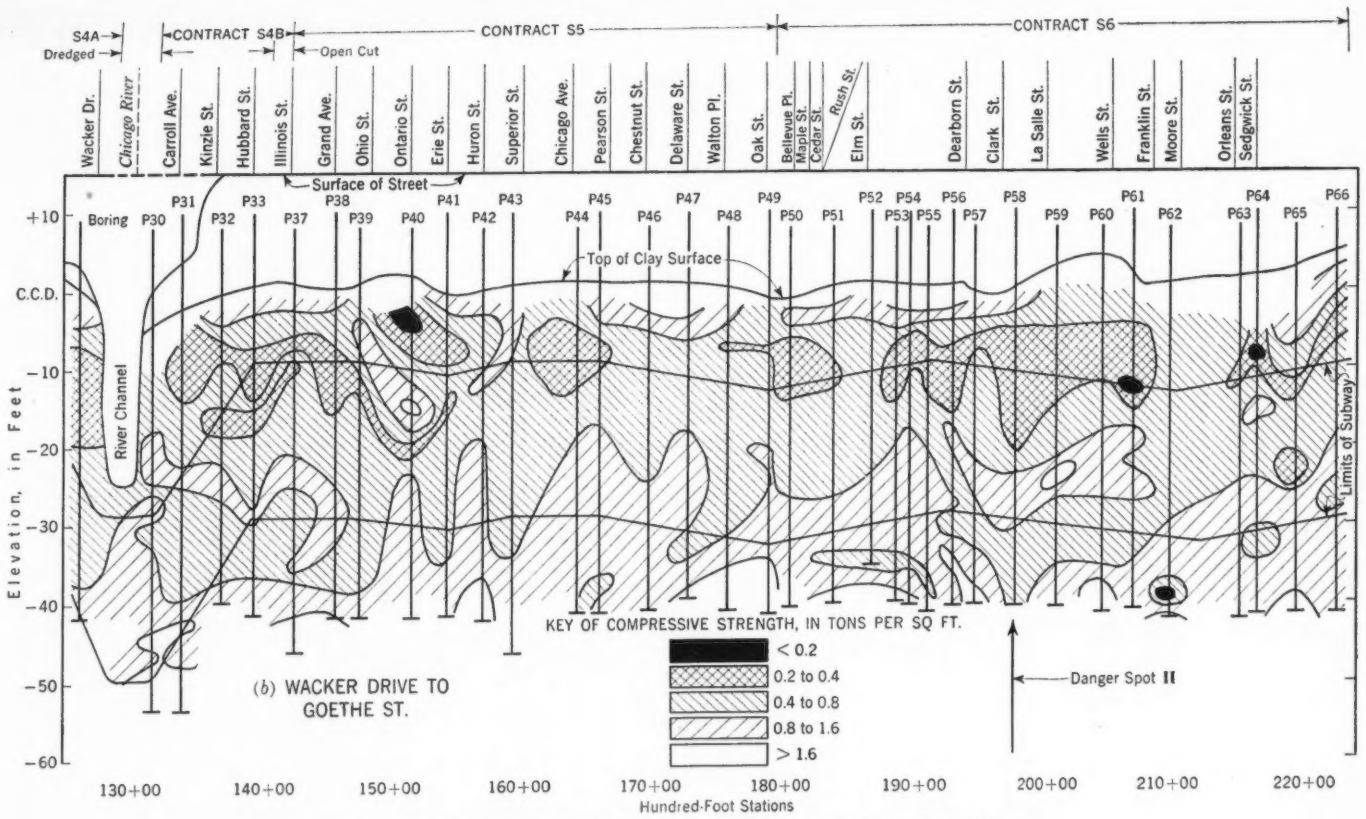


FIG. 8.—PROFILES, UNCONFINED COMPRESSIVE STRENGTH, ON THE STATE STREET ROUTE

the footings of the arch ribs. The factor of safety of the liner-plate support of the roof of the tunnel can be estimated by means of Eqs. 1 and 2.

Early in 1939, when the choice between liner-plate and shield method had to be made, the writer did not yet know to what extent the nonconfined compressive strength of seamless tube samples could be relied upon as a measure for the shearing resistance of the clay in connection with tunnel and open-cut problems. Nor did he know to what extent progressive failure is likely to influence the shearing resistance of the clay located above the tunnel. He had to depend on what he knew about the shearing resistance of soft clays from previous observations on clay slides. On the basis of this experience he assumed that this resistance is approximately equal to one half of the nonconfined compressive strength of seamless tube samples. On this assumption he made a rough estimate of the chances for the survival of a liner-plate support in the worst section of the subway. Since the assumption was not yet confirmed by experience, the writer did not attempt to use the results of his estimates as an argument in favor of the shield method on the southern routes. He merely advocated this method on general principles, without theoretical comments. Based on their construction experience, other members of the organization also insisted on using the shield method on the southern routes in spite of the expenditure involved. The final decision was reached on the basis of what amounted to a majority vote in favor of the shield on the southern routes.

During the succeeding two years the observations in open cuts demonstrated that the effective shearing resistance of the Chicago clay really is approximately equal to one half of the nonconfined compressive strength of the seamless tube samples. Hence the writer feels justified in recommending Eqs. 1 and 2 for general use in connection with tunneling in soft clay.

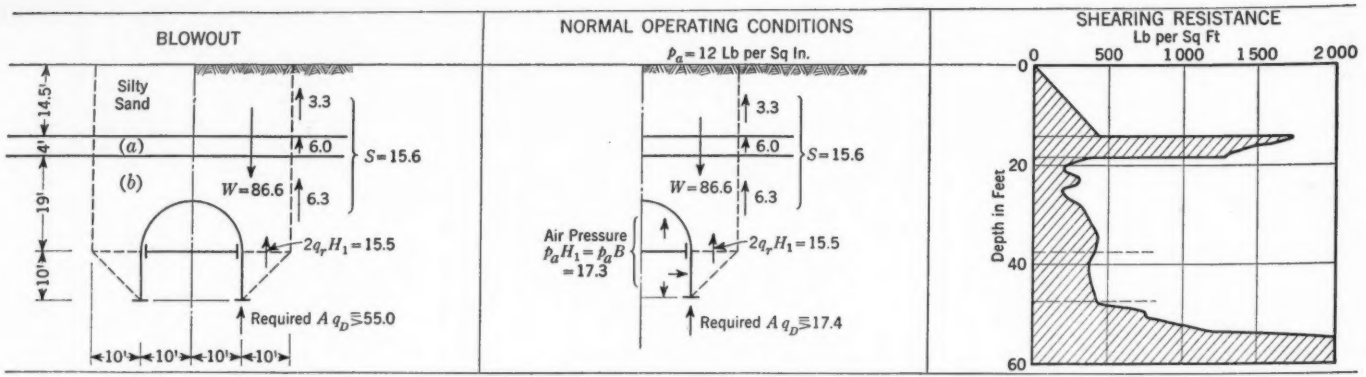
Danger-Spot Method for Deciding Between Liner-Plate and Shield.—On account of the great difference between the cost of the liner-plate and the shield tunnels and of the disastrous consequences of misjudgment, it is of great practical importance to make the choice independent of the personal equation. The following is an attempt to accomplish this purpose.

Since a blowout can occur at any stage of construction, the investigation needs to be made only for the weakest spots. These spots were located at the intersection of South State Street with 8th Street south of the Chicago River (danger spot I, Fig. 8(*a*)) and at the intersection between Division and Clark streets, north of the Chicago River (danger spot II, Fig. 8(*b*)). The conditions for the validity of Eqs. 1 (presence of an exceptionally stiff stratum on both sides of the tunnel) are not satisfied at either one of these two spots. Therefore, it is sufficient to consider the stability conditions expressed by Eqs. 2. From Eq. 2(*b*) one obtains, as a minimum requirement for the ultimate bearing capacity $A\,q_D$ per unit of length of one row of footings, the equation

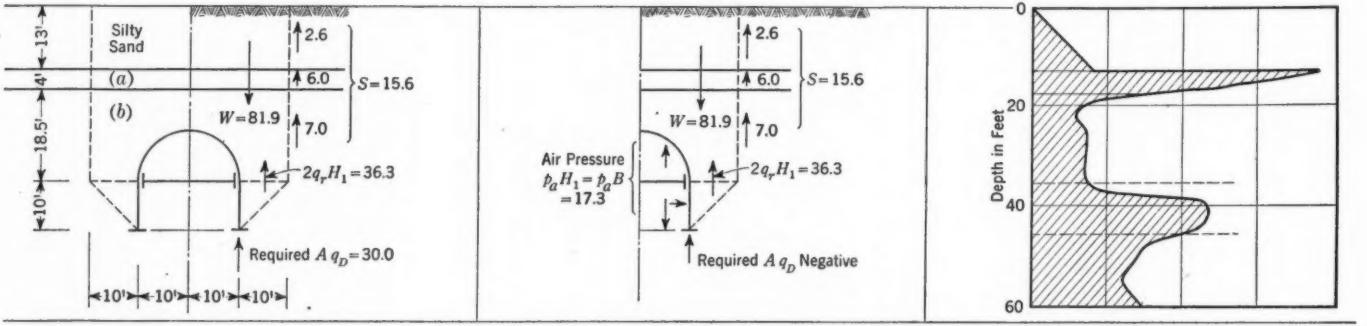
$$A\,q_D = W - S - 2\,q_r\,H_1 \qquad (3)$$

in which A is the area of the base of one row of footings per foot of tunnel, and q_D the ultimate bearing capacity of the supporting clay per square foot.

Fig. 9 shows several simplified sections through the soil and through the tunnel prior to placing the concrete. The unit weight of the soil within the



(a) DANGER SPOT I



(b) DANGER SPOT II

FIG. 9.—SIMPLIFIED SECTIONS PRIOR TO PLACING CONCRETE
(All loads and forces given in kips per foot)

zone of arching is roughly 120 lb per cu ft. The shearing force S in Eq. 3 is equal to the sum of the shearing resistance of the uppermost stratum chiefly composed of silty sand, of the hard clay stratum a and of that part of the soft clay stratum b which is above the level of the horizontal struts in the tunnel. The shearing resistance of the silty sand was computed on the assumption that the ratio between the horizontal and the vertical unit pressure at any depth is equal to 0.45, and that the angle of internal friction is equal to 30°. The effective shearing resistance of the clay is assumed to be approximately equal to one half of the nonconfined compressive strength of the seamless tube samples obtained from the test borings at the danger spots (in pounds per square foot) for the two danger spots. Under normal operating conditions the air pressure p_a in the tunnel was approximately equal to 12 lb per sq in., or 1,730 lb per sq ft in excess of the atmospheric pressure. By means of these data, Eq. 3 can be solved. The principal items are indicated in Fig. 9(*a*) under the heading "Blowout."

According to the results of the computation the value $A\ q_D$ per linear foot of one row of footings at danger spot I is equal to 55 kips and at danger spot II is equal to 30 kips. The largest footings used on the subway cover an area of 2 sq ft per ft. Hence, in the event of a complete blowout the footings would have to sustain a pressure of at least 16 tons per sq ft at danger spot I and 8 tons per sq ft at danger spot II, respectively. At neither site is the nonconfined compressive strength of the clay immediately beneath the base of the footings greater than 0.8 ton per sq ft. The corresponding ultimate bearing capacity q_D is about 2.0 or 2.5 tons per sq ft. Thus a liner-plate support could not sustain the clay pressure in the event of a complete blowout at either one of the two danger spots. However, on the southern route, similar unfavorable conditions prevail over almost the entire length of the route, whereas on the northern route they are limited to a section with a length of a few hundred feet.

For normal operating conditions the minimum requirement for the ultimate bearing capacity $A\ q_D$ per unit of length of one row of footings is determined by Eq. 2*a*. Transposing this equation to solve for $A\ q_D$—

$$A\ q_D = (W - S - B\ p_a) - (2\ q_r + p_a)\ H_1 - A\ p_a \quad \ldots\ldots\ldots (4)$$

Assuming p_a = 12 lb per sq in. = 1,730 lb per sq ft, and A = 2 sq ft per ft, one obtains, for the individual items on the right-hand side of this equation, the values indicated in Fig. 9 under the heading "Normal Operating Conditions." For danger spot I the value of $A\ q_D$ is equal to 17.4 kips per ft or 4.3 tons per sq ft of the footings and for danger spot II it is negative. Since the ultimate bearing capacity q_D of the clay at both spots ranged between 2.0 and 2.5 tons per sq ft, the temporary support of the roof would have failed on the southern route under normal operating conditions not only at danger spot I but also at several other places. Therefore, the decision to use the shield method on the southern routes was based on sound judgment. On the northern route the liner-plate method was adequate everywhere, under normal operating conditions, because it was even amply adequate at the danger spot II.

All the Chicago sewer tunnels were built successfully by the light liner-plate method. However, the width of most of these tunnels is considerably smaller than that of the subway tunnels and the width of the tunnels has an important influence on the success of the liner-plate method. By repeating the computation illustrated for "normal operating conditions" in Fig. 9(*a*), for a tunnel with a width and a height of 14 ft instead of 20 ft, the pressure on the footings is found to be equal to 1.8 tons per sq ft against 4.3 tons per sq ft. Since danger spot I represents the weakest spot on the entire line, a 14-ft tunnel could be constructed beneath South State Street by means of the liner-plate method at an air pressure of 12 lb per sq in., provided one could be certain that there is no blowout. The result of the computation demonstrates the danger of generalizing the experiences made when constructing tunnels through soft clay.

Influence of Size of Footings on the Safety of Liner-Plate Procedure.—In the preceding computations it was assumed that the area covered by the footings of each row is equal to 2 sq ft per ft. On the job the area of the footings ranged between 0.8 and 2 sq ft per ft. According to the blowout conditions illustrated in Fig. 9(*b*), the value of $A\ q_D$ (required bearing capacity for the footings) is negative for the danger spot II. Hence on the northern routes under normal operating conditions, the smallest footings are large enough. However, in the event of a serious blowout, the temporary support on the roof of the tunnel may fail only on account of inadequate size of the footings. Therefore it is advisable to specify, in the future, a lower limit for the size of the footings. This size can be computed for each contract by means of Eq. 3 on the basis of the least favorable conditions disclosed by the soil profile.

IV.—Clay Pressure After Concreting and the Design of Tunnel Tubes

Purpose of Investigations.—The writer's conceptions regarding the pressure of the clay on the tunnel tubes are stated in Section I. To verify or modify these conceptions, the Subway Department decided to determine, by direct measurement, the change in the shape of a fairly flexible experimental tube designed on the basis of an arbitrarily selected value for the ratio between the horizontal and vertical pressure in the clay.

According to the results of experimental investigations in the laboratory, it appears likely that the ratio K_0 between the horizontal and the vertical unit pressure at a given point in a sedimentary deposit of sand is of the order of magnitude of 0.45. In a normally consolidated bed of clay such as the soft blue clay in Chicago, this ratio is likely to range between 2/3 and 7/8. In both types of soil it is less than unity.

During the process of mining, the horizontal pressure in the clay on both sides of the tunnel decreases from its normal value to the value of the air pressure that represented the average equivalent of the weight of a 15-ft layer of clay. The overburden had an average depth of 35 ft. On the other hand, immediately beyond the zone of arching represented by a shaded area in Fig. 3(*b*) the vertical pressure is considerably greater than the normal overburden pressure because the clay beyond this zone carries both the weight of the overburden and part of the weight of the clay above the tunnel. Hence

the process of mining tends to reduce the average value of the ratio from K_0 to a smaller value K_1.

When the temporary support of the clay is replaced by a relatively rigid tunnel tube, the average value K gradually changes and approaches a final value K_2. There is no reason for assuming that K_2 should be greater than K_0 which ranges between 2/3 and 7/8. It is most likely that K_2 is approximately 2/3.

To establish an important difference between the stress conditions which exist in the vicinity of a rigid tube and those imposed upon the clay by a flexible tube, it was decided to give the flexible experimental tube a circular cross section. If it is subjected to an equal all-around pressure, the shape of the tube remains unchanged. On the other hand, if the horizontal pressure in the clay is less than the vertical pressure, the tube flattens until a similar state of stress is established. A tube with very thick walls is likely to fracture during

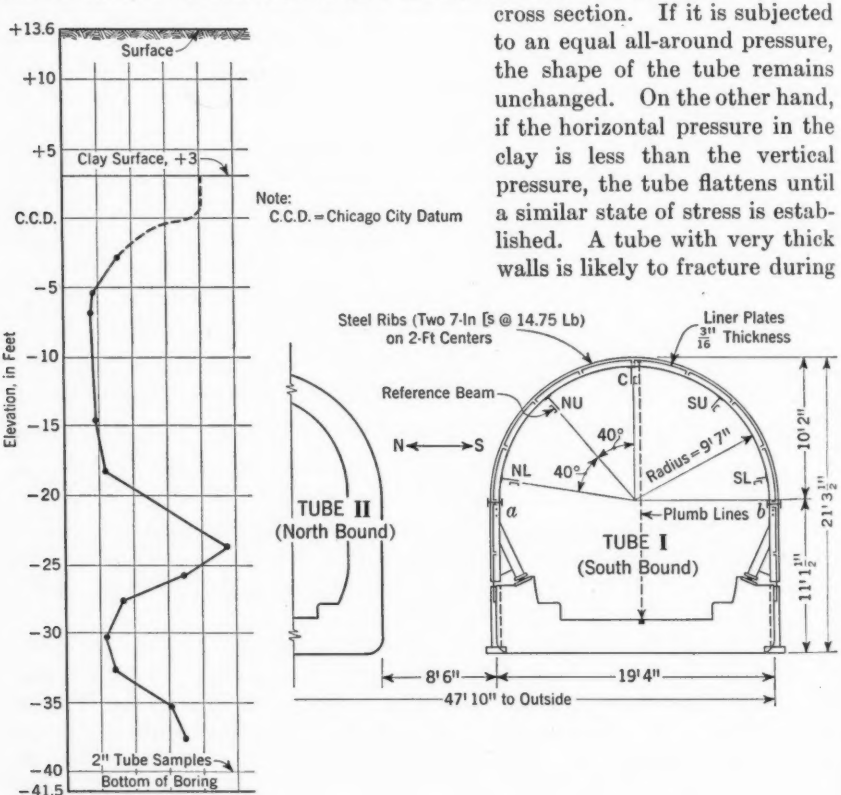


FIG. 10.—DISTORTION MEASUREMENTS, EXPERIMENTAL SECTION S-6

the process of adjustment unless it is heavily reinforced, whereas a tube with thin walls undergoes only elastic deformations. To design a tube in such a manner that it does not break necessitates knowing the magnitude of the deformations involved. Therefore the core of the investigations was

the measurement of the deformations. As a by-product, information on the intensity and the distribution of the contact pressure on the base of the invert was obtained.

Experimental Section on Contract S-6.—The experimental section, shown in Fig. 10, is located on contract S-6 between Station 201+47 and Station 201+68 in the southbound tube, which was the first constructed. It has a length of 21 ft. Above the wall beams the tube is composed of steel ribs on 2-ft centers curved to an inside radius of 9 ft 7 in., with steel liner-plates bolted to the webs of the ribs to span the space between the ribs. The manner of

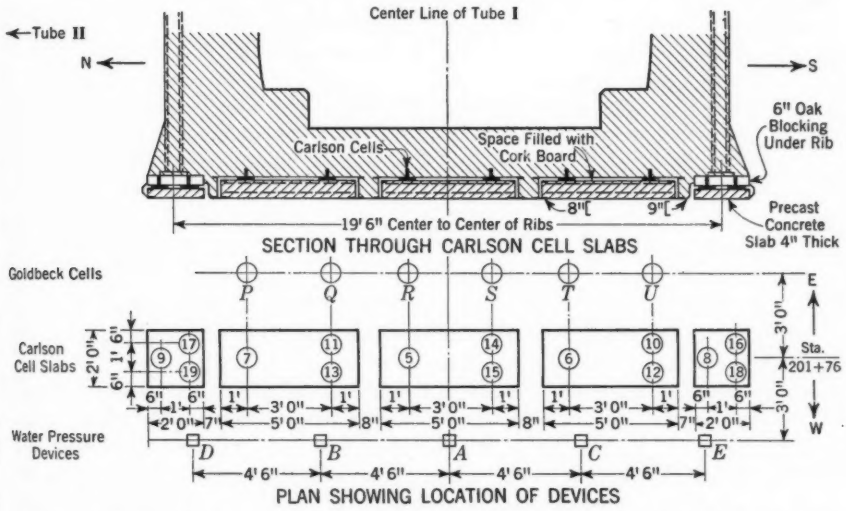


FIG. 11.—LOCATION OF PRESSURE MEASURING DEVICES, EXPERIMENTAL SECTION, CONTRACT S-6

erecting the ribs and of the liner-plates was the same as that for contracts where ribs and large liner-plates were used for temporary lining. The ribs were composed of two 7-in. channels riveted together back to back in order to provide sufficient area to carry the direct compression at a moderate unit stress. The plates were fabricated from 3/16-in. sheet steel.

Below the wall beam, normal construction procedure involves the erection of vertical steel ribs supported on wood bearing blocks. To avoid complications in the erection procedure, this method was adhered to and the concrete invert was poured as usual. The brackets were subsequently fastened to the vertical ribs below the wall beams. The hinge was supported on the concrete invert. After erection of the brackets, the vertical steel ribs were burned through above the invert. The space between the liner-plates and the clay was filled with wood blocking grouted with sand-cement grout.

To measure the distortion of the ribs, five steel reference beams were located as shown in Fig. 10. They are designated by NL, NU, C, SU, and SL. The ends of the reference beams were supported by steel sleeves embedded in the concrete of the adjacent permanently-lined sections. The action of the

experimental section was made independent of the adjacent concrete sections by means of sliding joints. The distance between the ribs and the reference beams was measured between two gage points by means of a pair of calipers and a steel scale reading directly to 0.02 in.

In addition to the distortion measurements, provision was made for strain readings on the ribs themselves. On the circular section above the wall beam, eight 2-in. and four 10-in. gage lines were located on the center line of each rib to determine the compressive stress in the ribs. Many others were located on the ribs below the wall beam. In all, 840 strain-gage lines were established. The holes were drilled and initial readings were made while the steel was still located at the fabricating plant. Readings were made with a 2-in. and a 10-in. Whittemore strain gage, using a mild steel standard bar to eliminate the effect of temperature strains.

The principal features of the invert section are shown in Fig. 11. The center line of this section is at Station 201+76, approximately 8 ft from the end of the arch experimental section, in a part of the tunnel fully lined with concrete in the customary manner. Carlson cells were used to measure the contact pressure between the bottom of the invert and the underlying soil. In contrast to the readings on the Goldbeck cells, those on Carlson cells can be made without previously displacing one of the constituents of the cell with reference to the others. The cell acts like a thin slab, whose modulus of elasticity in a direction at a right angle to the contact face is almost as high as that of concrete. As a consequence the deformation of the cell associated with a change of the contact pressure is negligible for most practical purposes.

The bearing area of the Carlson cell is 30 sq in. as compared with 10 sq in. for the Goldbeck cell. Nevertheless, it is by far too small for obtaining (by any reasonable number of cells) a fair average of the contact pressure on the base of an invert with a width of about 20 ft, supported by the natural ground. In order to overcome this difficulty, the writer proposed constructing larger measuring units that consist of heavily reinforced, rectangular concrete slabs. The dimensions of these units are given in Fig. 11. Each unit rests on the natural ground and the intensity of the contact pressure on each unit is measured by means of three Carlson cells located between the top surface of the concrete slab and the concrete invert. Two of the units, placed beneath the steel ribs, are 2 ft by 2 ft in plan and were pre-cast so that the entire assembly of the slab and the cells could be set beneath the rib instead of the wooden bearing block customarily used. Great care was taken to prevent the concrete from entering the assembly. The remaining slabs were 2 ft by 5 ft in plan. They were cast in place on the invert soil in channel forms 8 in. deep. Surrounding each slab was a second channel frame 9 in. deep with a clearance between the two frames of 1/2 in. This space was filled with porous cork board to prevent the entrance of concrete. On top of the measuring slabs, steel plates were bedded at the location of the Carlson cells, and the cells were seated on the plates with plaster of Paris. The part of the slab not covered by Carlson cells was built up to a height of the outer channel frame with cork board and covered with a metal sheet to prevent the invert concrete from coming into contact with the slabs except over the cells. Cork board

was chosen because it was sufficiently rigid to compress only 0.1 in. under the weight of the fresh invert concrete, but sufficiently compressible to carry no appreciable part of the load after the concrete had set.

Because of the wide previous use of Goldbeck cells, it was considered desirable to make an installation of these devices in the invert section and to compare the readings with those obtained by means of the Carlson cells and the strain gage readings in the ribs. Accordingly, one Goldbeck cell was placed opposite each Carlson cell location in the central part of the invert as indicated in Fig. 11 (cells P to U). Each Goldbeck cell was attached to a length of flexible high-pressure tubing in the laboratory and all connections made, so that no piping or field connections were necessary. Each cell was placed face down on the clay surface and carefully bedded. It was then anchored in position and eventually covered by the invert concrete.

Finally a group of five water-pressure measuring devices was installed for the purpose of measuring the hydrostatic pressure in the water contained in the clay below the invert.

Results of Observations.—During the period of observation the following events are of special importance:

Date (1940)	Description	Cumulative time t, in days
February 27	Erection of arch section started	....
February 29	Invert section completed, initial readings on Carlson cells	....
March 20	Initial readings on reference beams for deformation measurement	0
March 23	Hinges in arch section freed	3
March 27	Working face of the second tube passed by the Experimental Section located in the first tube	8
March 30	Temporary drop of air pressure from 11.5 to 8 lb per sq in. while the second tube was not yet concreted	10
July 19	Air pressure removed	121

Fig. 12 shows the deformation of the tube at different stages before and after the removal of the compressed air. During the process of mining, the arching above the roof of the tunnel increases the vertical pressure in the clay in the vicinity of the boundaries of the zone of arching over its normal value. While the second tube was mined part of the weight of the clay located above this tube was transferred to the north half of the experimental section. The increase of the vertical clay pressure on this part caused by mining the second tube produced the unsymmetrical deformation of the experimental section, shown in Fig. 12.

Fig. 13 represents the relation between the time and the horizontal displacement of the points NL and SL (Fig. 10), respectively. Both points moved outward, which indicates that the initial ratio between the horizontal and the vertical pressure in the clay was smaller than unity. A considerable

part of this movement occurred at time $t = 8$ days, while the second tube was being mined. Since the second tube was located on the north side of the experimental section the point NL moved much more than the point SL. After a period of about 500 days both points became practically stationary. The final displacement of NL was 0.375 in., and that of SL was 0.210 in.

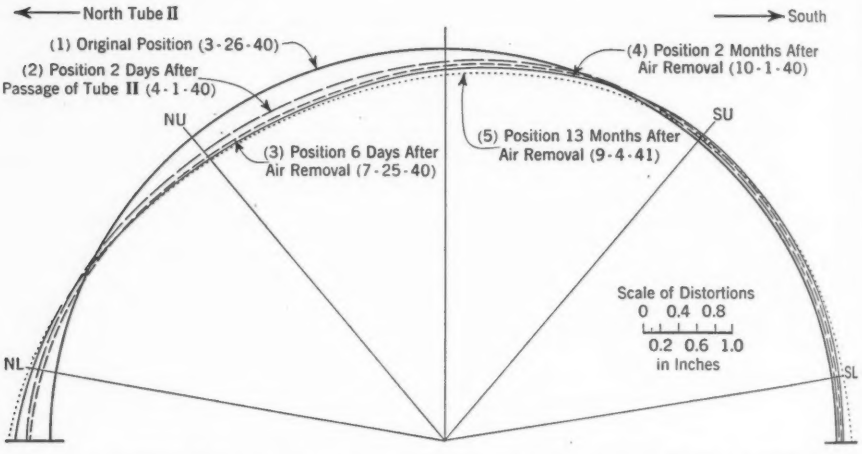


FIG. 12.—DISTORTION MEASUREMENTS, EXPERIMENTAL SECTION, SOUTHBOUND TUBE I, CONTRACT S-6

Hence the horizontal diameter of the tube did not increase by more than 0.85 in., which is an insignificant amount and much smaller than the writer anticipated. Almost one half of the increase occurred during the first two weeks after the hinges were freed.

The results of the pressure-cell readings are shown in Fig. 14. The two horizontal lines represent ideal cell readings computed on the assumption that there is no arching above the tunnel and that the contact pressure is uniformly distributed over the base of the invert. They correspond to the states before

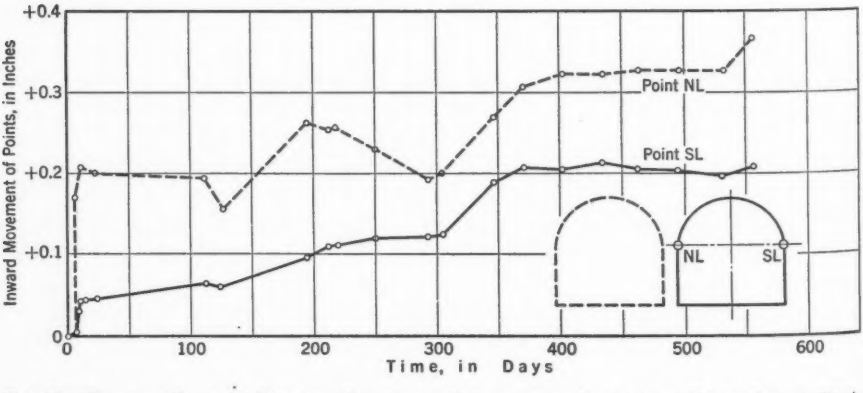


FIG. 13.—RELATION BETWEEN TIME AND HORIZONTAL DISPLACEMENT (AVERAGE OF THREE CENTER RIBS)

and after the removal of the air pressure respectively. At the end of the first month after the section was mined, the arching still carried a considerable part of the weight of the overburden. This is shown by the position of the line marked 3–26–40 with reference to the upper horizontal line. However, as time went on the arching effect decreased and the contact pressure approached

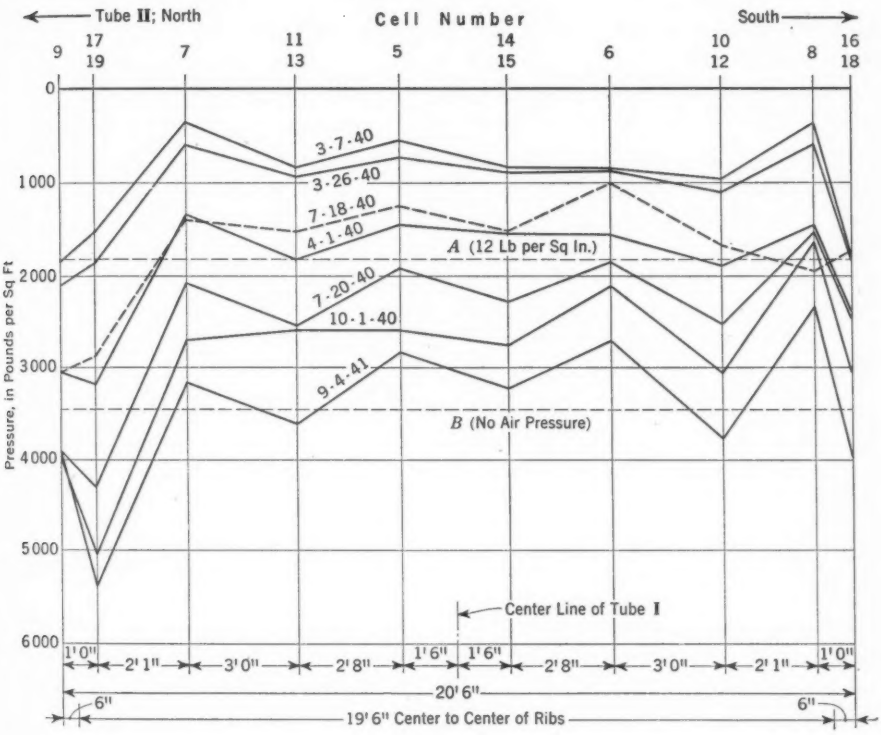


FIG. 14.—INVERT PRESSURE MEASUREMENTS, CARLSON CELL SLABS, EXPERIMENTAL SECTION, CONTRACT S-6

the difference between the weight of the overburden and the air pressure. Removal of the air induced a temporary revival of the arching effect. The final value is again fairly close to the weight of the overburden. Along the outer edges of the invert the unit contact pressure is somewhat greater than the average contact pressure on the central part of the invert. This is chiefly a consequence of establishing the footings for the arch ribs before the invert is poured. While the second tube was mined, the unit pressure along the northern edge of the tube became permanently greater than that along the southern edge. The reason is identical with that for the unsymmetrical deformation of the rib. However, all these deviations from the uniform pressure distribution are not important enough to deserve practical consideration.

All the Goldbeck cells remained in operation and gave fairly consistent results. However, the individual readings are so scattered that they give no

indication of the true distribution of the contact pressure as disclosed by the Carlson cells.

The following data represent the summary of the results of measurements of the contact pressure made in September, 1940:

Method	Stress, in kips per ft
Strain gage, assuming that liner-plates act with the ribs	66
Carlson cells	66.0
Goldbeck cells	106.5
Computed overburden 118 lb per cu ft	72.6

The water-pressure measuring devices still fail to indicate a pressure in the water below the invert, although the base of the invert is located at a depth of about 30 ft below the free water table and at about the same depth below the lake level. This fact and the results of other independent observations seem to indicate that the tunnels act as drains in spite of the thickness of the walls and the liner-plates which cover the outside of the tubes.

Application of Test Results to the Design of Tunnel Tubes.—As stated in Section I, the design of the tunnel tubes was based on the following assumptions:

(1) The ratio K between the horizontal and the vertical unit pressure has the same value at any point of the clay;

(2) K may have any value between 1/3 and 2/3; and

(3) The presence of the tubes has no influence on the state of stress in the clay.

The most economical method of designing in accordance with these assumptions is the following: The arch is given such a shape that the bending moments are equal to zero for some value of K intermediate between 1/3 and 2/3. Then one computes the bending moments on the assumption that K is equal to 1/3 and 2/3, respectively. Finally, one selects the thickness of the arch and the amount of reinforcement such that, for both extreme values of K, the stresses in the walls of the tube are within the allowable limits.

This or a similar method of computation led to the result shown on the left-hand side of Fig. 15(*a*). At the crown the arch has a thickness of about 2 ft, at midheight a thickness of 2.5 ft, and at the level of the rails a thickness of more than 3 ft. It is heavily reinforced.

The observations in the experimental section on contract S-6 have shown that a very small change of the horizontal diameter of the tunnel tube is sufficient to change the state of stress in the clay in such a manner that no stiff walls are required to withstand the pressure of the clay, provided the following conditions are satisfied: First, the shape of the arch should be such that the bending moments in the arch are equal to zero for any arbitrary value of K intermediate between unity and 2/3 or less. Second, the arch should be thin enough to follow the deformation of the clay without rupture. However, even a local failure does not endanger the tunnel. Railway tunnels with thick walls have survived a settlement at the crown of more than 1 ft and an irregular distortion of their shape. In the case under consideration the

deformations do not exceed a fraction of an inch. These empirical facts make it possible to reduce the quantity of concrete and of reinforcement for the arch to a fraction of what is required according to the customary methods of design and, in addition, to reduce the quantity of excavation quite substantially.

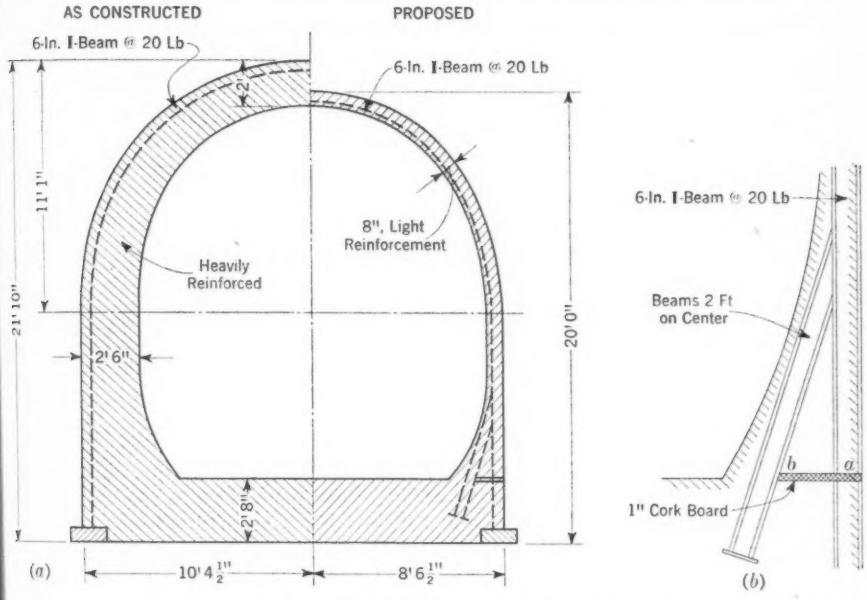


FIG. 15.—COMPARISON OF PROPOSED AND ACTUAL DESIGNS

The right-hand side of Fig. 15(*a*) illustrates one of several possibilities for taking advantage of the foregoing conditions. The arch shown in the figure has a thickness not exceeding 8 or 10 in. No harm is done if the shell is only 6 in. thick in some places because the steel ribs alone are sufficient to carry the pressure of the clay. The mining is done by using the light liner-plate method without any modification. The ribs (6-in., 20-lb I-beams) have a slightly elliptical shape. They do not differ from those used before except that they are provided with brackets as shown in Fig. 15(*b*). After the tunnel is mined the invert C_1 is concreted. The lower ends of the brackets are buried in concrete. After the concrete of the invert has set, the vertical legs are burned at point a whereby the pressure due to the overburden is transferred on to the brackets. Then the surface of the concrete between a and b is covered with cork board, to provide some flexibility for the connection between the arch and the invert. Finally the arch is poured.

It has been mentioned that almost one half of the deformation of the experimental section occurred within the first two weeks after the hinges were freed. The subsequent increase of the horizontal diameter of the tunnel did not exceed 0.3 in. Therefore the concreting of the arch should be postponed until the second tube has been mined or, preferably, until the air is turned off.

Once the ends of the brackets are buried in the concrete, even a complete blowout has no more harmful effect, because the invert is designed to carry the weight of the entire overburden, without the assistance of compressed air.

One might object that a tunnel tube with thin walls may be less likely to suffer the consequences of a subsequent change in the state of stress in the clay (for instance, due to excavation in the vicinity, or the construction of a building with shallow foundations) than a tube with thick walls. In the writer's opinion, a tube with thick walls is much more sensitive to a change in the state of stress in the clay than one with thin walls. When supported by the walls of a flexible tube without any gap, the clay is very much stronger than any reasonable amount of reinforced concrete which may be used for strengthening the tube.

The difference between the two designs illustrated by Figs. 15(*a*) and 15(*b*), respectively, can briefly be described as follows: In the case represented by Fig. 15(*a*) the designer ignores the virtues of the clay and must pay for it. In the other case, Fig. 15(*b*), the designer trusts the clay and the clay deserves the confidence.

In 1938, when the Chicago tunnel tubes were designed, it would have been reckless to propose the design shown in Fig. 15(*a*), because nothing positive was known regarding the magnitude of the deformations required to establish, permanently, an arbitrarily chosen state of stress in the clay. The method of computation that was used was the only one which could be relied upon. However, on account of the fact disclosed by observation in the experimental section the method of design illustrated by Fig. 15(*b*) becomes worthy of serious consideration. A tunnel tube with 8-in. walls can withstand, without serious harm, deformations ten times greater than those observed. Furthermore, the deformations of a tube with a slightly elliptical cross section can be expected to be smaller than those of a tube with a circular section.

Since the clay is not uniform the shape which the section of a flexible tube tends to assume changes from place to place in an unpredictable manner. However, the character of the clay changes very gradually both in a horizontal and in a vertical direction and the concrete is capable of very considerable plastic deformation without losing its strength. Therefore, the writer believes that the variable character of the lateral support of the tunnel tube is a factor of minor importance as long as the magnitude of the change of the horizontal diameter remains within the limits disclosed by the observations in the experimental section.

It goes without saying that Fig. 15(*b*) merely serves to illustrate the existing theoretical possibilities. In practice the imperfections of the methods of construction must be taken into consideration. They place a lower limit on the thickness of the walls of the tunnel.

A second experimental section was established in a shield tunnel, between Station 87+55 and Station 88+00 on contract S-3 (Fig. 1) (see Footnote 2*a*). The results of the observations in this section were at least as encouraging as those already described. The cost of both experimental sections and of the observations to date amounted to about $35,000—or 45¢ per linear foot of

single-track subway. A considerable part of the money was spent on concreting these sections after construction was finished, which was not included in the writer's program. As a matter of fact, it would have been a great asset to engineering science if permission had been granted to keep the experimental sections permanently accessible. By taking advantage of the facts disclosed by the observations, the cost of the tunnels could be reduced by at least $30 per ft.

V.—Settlement of Street Surface and the Rôle of the Squeeze Observations

Settlement Observations.—The program for the settlement observations included, among others, the following items:

(1) Establishment of level points on the buildings along the routes;
(2) Establishment of level points on the street surface above the subway on profiles spaced 20 ft;
(3) Daily readings on all level points in the vicinity of the mining operations;
(4) Measurement of lateral movements of adjacent buildings; and
(5) Construction of plots showing the level readings and a tabulation of the daily movements.

The bench marks were established beyond the range of possible movements due to mining. Three readings were taken with reference to the city datum and the mean was used as the elevation of the bench. Level marks were then placed on all columns, walls, machinery, or other important parts of the buildings. The initial elevations of these marks were checked in the same manner as the benches.

In the vicinity of the seat of mining operations, readings were taken daily or as often as needed. At times readings were made every two hours for a 24-hr period. Level parties were on call at all times to take care of emergencies. Experience showed that the settlement does not extend beyond a distance of about 75 ft from the lot line on each side of the street. Hence few readings were made beyond this distance.

The results of the settlement observations were condensed into settlement profiles containing the maximum settlement of the street surface due to mining for each of the two tubes individually, the maximum total settlement 3 to 20 days after removal of air pressure, the air pressure at the time of mining, the distance between the end of the finished tube and the working face, and finally general information concerning the soil conditions. The files of the Subway Department also contain complete settlement profiles on 20-ft intervals. The cost of the settlement observations amounted to $2.75 per linear foot of single-track tunnel.

General Characteristics of the Street Settlement.—Figs. 16(*a*) and 16(*b*) represent characteristic settlement profiles of the street surface above a twin tube, and Figs. 16(*c*) and 16(*d*) are similar profiles taken above double tubes. In each figure the ordinates of the lower curve represent the settlement due to the construction of the first tube (I = settlement until the arch was concreted

and I′ = settlement until the working face of the second tube approached the profile). The ordinates of the upper curves represent the supplementary settlement due to the construction of the second tube. (Curve II is the settlement between the time when the working face approached the profile and the time when the arch was concreted; and II′ is the settlement due to

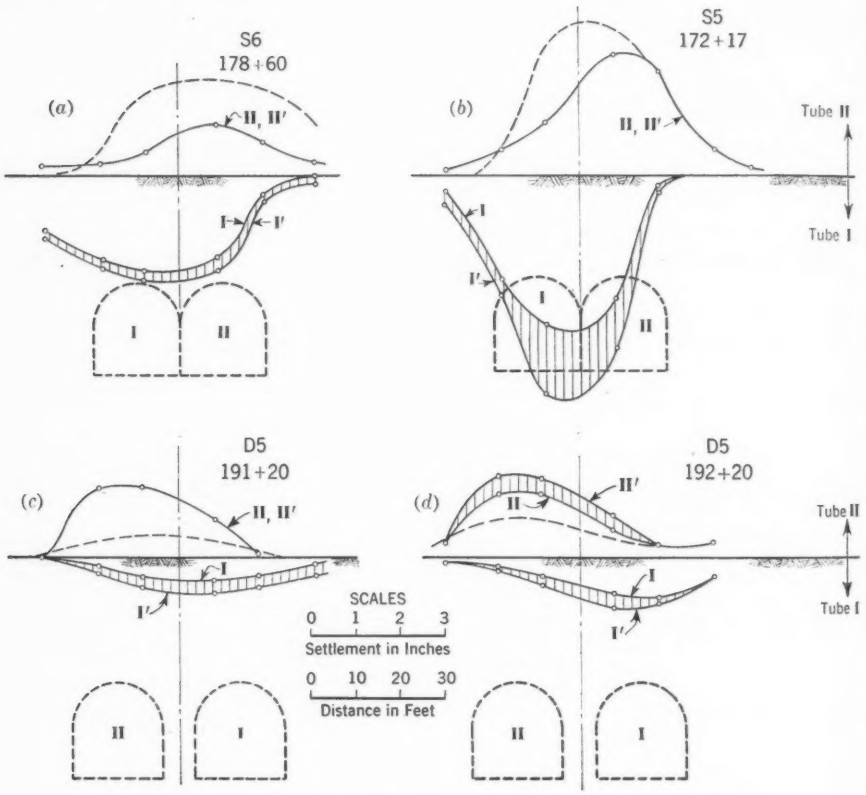


FIG. 16.—CHARACTERISTIC SETTLEMENT PROFILES OF THE STREET SURFACE

second tube until the air was turned off.) The broken-line curves represent reversed images of the curves I. They facilitate a comparison of the settlement due to mining the first tube with that produced by mining the second tube. If the movements produced by these two mining operations were identical, the broken-line curve should be identical with curve II.

The profiles disclose the following general characteristics of the settlement: The greatest settlement due to mining the first tube usually occurs near the middle of the street and not above the center line of the tunnel. This seems to indicate that the change of stress in the sandy top layer increases the susceptibility of this layer to being compacted by the street traffic. This interpretation is corroborated by the observation that the settlement of the street surface is commonly greater than that of the clay surface underlying

the sand. On the other hand, the greatest settlement due to mining the second tunnel is usually above the center line of this tunnel because at the time the second tunnel was mined the sand was already compacted.

The greatest settlement due to mining the second tube on a twin tube section is almost invariably much smaller than that due to mining the first one. Figs. 16(*a*) and 16(*b*) (and subsequently, Fig. 18) serve as examples. If the subway consists of a double tube the settlement over the second tube is everywhere very much greater than that caused by mining the first tube, as shown in Figs. 16(*c*) and 16(*d*). These rather surprising phenomena seem to be associated with the archings above the tunnel. If the second tunnel of a twin tube is mined, one side of the zone of arching is located above the first tube which represents a practically rigid support. On the other hand, while the second tunnel of a double tube is mined, one of two supports of the mass of clay which arches over the tunnel consists of a block of clay that has been disturbed while mining the first tube. Every disturbance inevitably reduces the rigidity of the clay. Hence twin tubes appear to be preferable to double tubes.

Squeeze Measurements.—The settlement of the street surface and that of the adjoining buildings with shallow footings represent an undesirable but inevitable accompaniment of tunneling in general. Even the expensive shield method cannot eliminate this feature. To be able to discriminate between avoidable and unavoidable settlement, one must examine the individual factors that contribute toward the settlement. This was accomplished by measuring the different components of the movement of the clay toward the tunnel. For future reference the measurements were supplemented by recording all the data that may help to identify the conditions under which the observations were made.

The following is a list of the items that constituted a complete set of squeeze measurements:

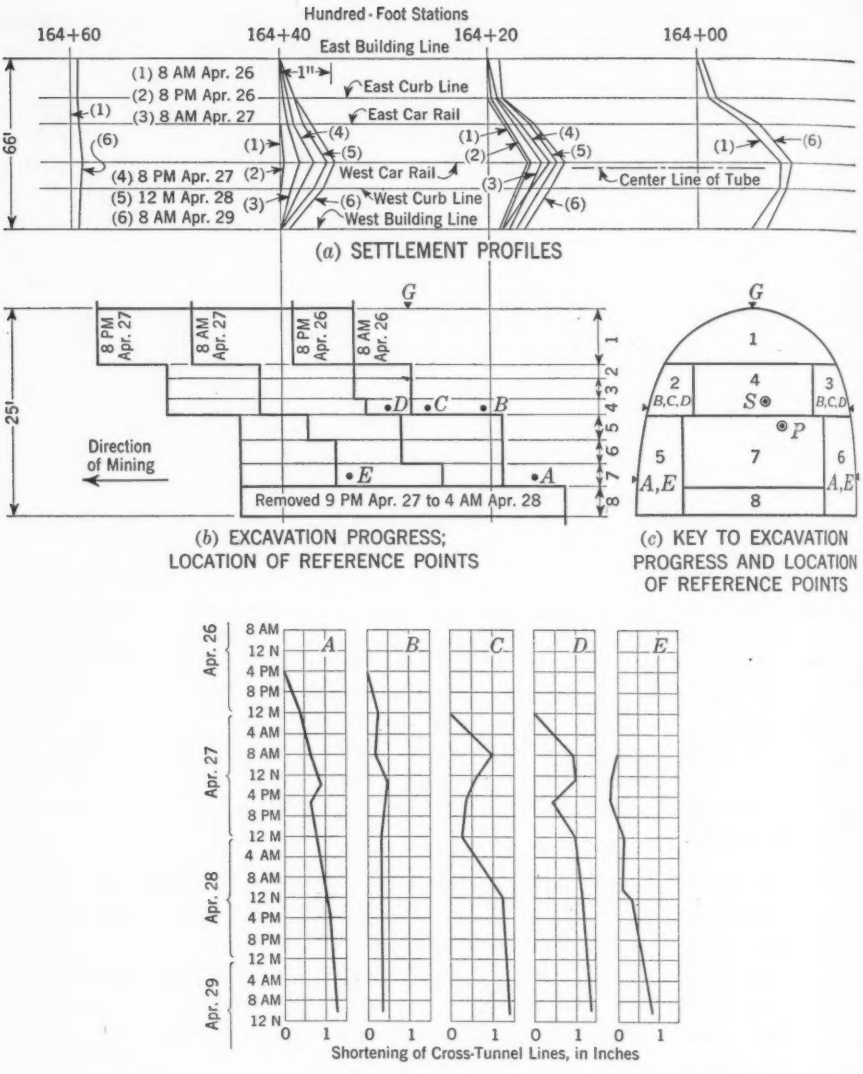
(*a*) Complete Record of Excavation and Construction Progress.—This record consisted of an hourly tabulation of the location of each of the portions of working face and other pertinent data.

(*b*) Measurement of Lateral Inward Squeeze of Clay Walls.—As soon as the clay on the sides of the tunnel was exposed by excavation, a steel rod was driven into the clay on each side of the tunnel and the distance between the rods was measured at suitable time intervals. Simultaneously the lateral movement of the legs of the arch ribs was observed and recorded.

(*c*) Downward Movement of the Tunnel Crown.—A steel rod was driven in a vertical direction into the clay in the roof and the elevation of the lower end of the rod was determined at suitable time intervals with a level.

(*d*) Settlement of the Lining.—Level readings were made on the wall beams and on the footing blocks in order to determine whether or not vertical movement of the temporary lining occurred.

(*e*) Inward Movement of the Working Face.—Steel spearheads were set in advance of the face before mining operations in a given heading were undertaken. These spearheads were driven into the working face to a distance up



Point (Fig. 17(*c*))	Location	Time of setting	Settlement (in.)
G	Crown reference points	11:00 a.m.; April 26[b]	2¼
S	Spearheads; No. 1 set 10 ft beyond face	3:00 p.m.; April 25	1¼[a]
	No. 2 set, during mining, 9 ft beyond face	1:00 p.m.; April 29	1[a]
P	At working face after mining	No movement during 24 hr while face was standing	

[a] Toward excavation during mining. [b] Read at noon, April 29.

FIG. 17.—RECORDS OF SQUEEZE TEST, SOUTHBOUND TUBE, APRIL 26–29, 1939

to 12 ft ahead of excavation by means of a 1-in. pipe. A smaller steel rod which could be inserted inside the pipe was then used to push the spearhead away from the end of the pipe and leave it surrounded on all sides by clay. After the spearhead was ejected from the pipe, but while the rod was still in contact with its rear end, the distance between the spearhead and a reference point in the completed part of the tunnel was measured. A second measurement was made when the spearhead was uncovered by excavation. The difference between the two measurements is equal to the inward movement of the spearhead during mining. In one instance the spearhead was connected with a steel wire that protruded beyond the original position of the working face. Thus it was possible to ascertain the position of the spearhead at different stages of mining.

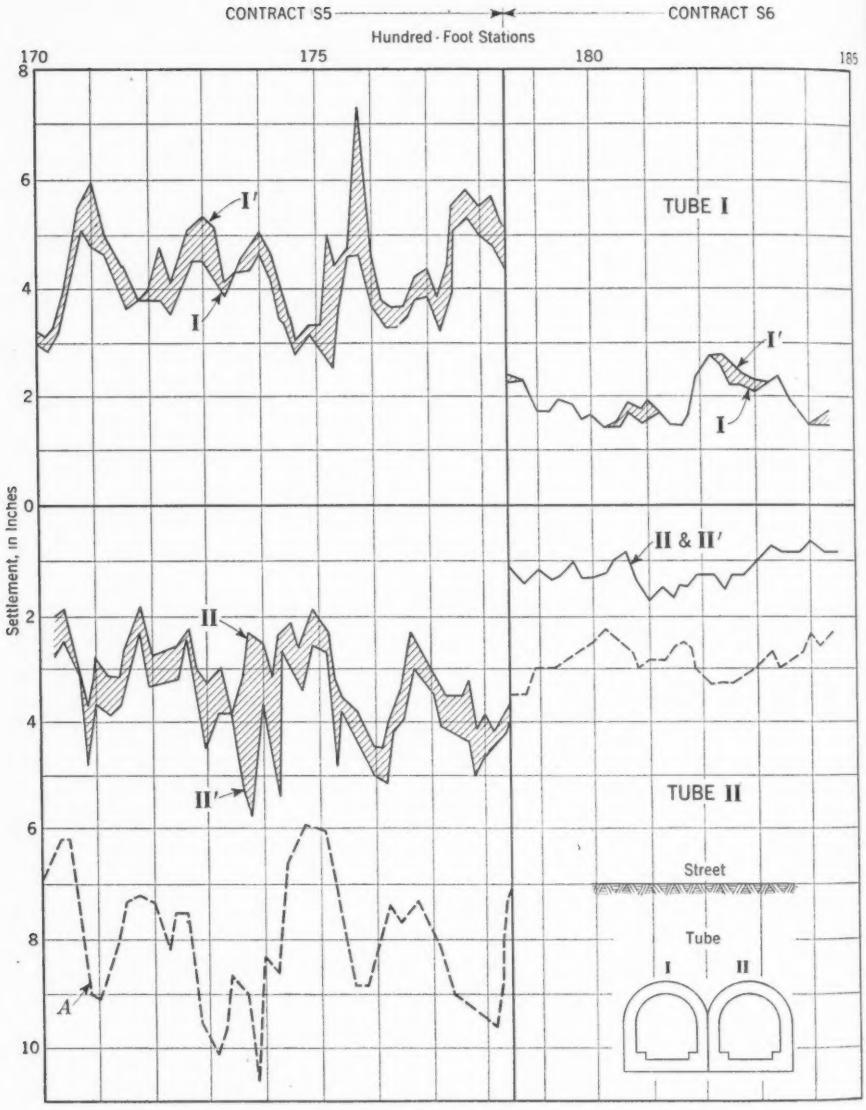
(*f*) Soil Characteristics.—The regular boring data were supplemented at the localities where squeeze measurements were made by soil tests on numerous soil samples taken from the heading during the period of observation.

(*g*) Air Pressure, Etc.—A record of the variations of the air pressure was obtained from the recording gage at the engineer's office. All other construction details that might have an influence on the measured movements were carefully observed and recorded.

Fig. 17 represents the result of a squeeze measurement on contract S-5 which was the first contract awarded. Fig. 17(*a*) is a plan of the street located above the place where the squeeze measurements were made. The reference points for settlement observations are located on transverse lines and the settlements are plotted from these lines in a horizontal direction toward the right. As a result of this and of subsequent tests on this contract, it was found that part of the settlement above the tunnel was due to a lateral movement of the clay toward the tunnel and part of it was due to the existence of empty spaces between clay and liner-plates. Hence on the subsequent contracts efforts were made to reduce the importance of these two sources of settlement. These efforts were remarkably successful.

Influence of Construction Procedure on the Settlement of the Street Surface.—A conspicuous example of the influence of the details of construction on the settlement is shown in Fig. 18, which represents the settlement of the street surface on both sides of the boundary between contracts S-5 and S-6 on North State Street.

Contract S-5 was the first to be awarded. The contractor selected a method of construction which had been used successfully on the construction of the largest of the sewer tunnels in the Chicago area. Mining was done without the aid of a wall beam, by means of the procedure illustrated by Fig. 2(*a*). However, each operation of mining the side pockets and erecting rib segments below already completed portions removed most of the support afforded by the rib to the clay overlying the tunnel. As a result it was frequently found that the crown segment settled as often as a lower portion was added to the rib. By the time the footing blocks at the base of the ribs were placed, the ribs had undergone a considerable downward movement. As a consequence, on numerous occasions, some crown ribs were at a considerably



Curve	Identity; Maximum:
I	Settlement prior to pouring concrete in Tube I, southbound
I'	Settlement prior to the approach of Tube II, northbound
II	Increase in settlement between the approach of Tube II and the beginning of concreting
II'	Increase in settlement between the passing of Tube II and the removal of air pressure
A	Total settlement 12 to 25 days after the removal of air pressure

FIG. 18.—SETTLEMENT AS INFLUENCED BY DETAILS OF CONSTRUCTION

lower elevation than others. The size of the footings was equal to 1 sq ft per ft of the length of the row. Sand-cement grout was injected through holes in the liner-plates from an air line. One gained the impression that the grout did not penetrate all the empty spaces. However, the influence of these details of construction on the settlement of the street surface was not known because in former days no systematic records of the settlement of the street surface were kept. Therefore, it was decided to investigate this influence and to modify the method of construction on the later contracts in accordance with the results of the observations.

Once the relations between the underground operations and the settlement of the street surface were clearly recognized it was a relatively simple matter to reduce the settlement by appropriately modifying the procedures on the subsequent contracts, including the adjoining contract S-6. One step consisted of inserting a wall beam between the curved and the straight part of the arch ribs, approximately at midheight of the tunnel. The wall beams prevented the settlement of the crown. By bracing the wall beams against each other with horizontal struts it was also possible to reduce the lateral movement of the clay toward the tunnel. Another important step consisted in changing the method of grouting. On contract S-6, instead of attempting to fill the space between clay and liner-plates with sand-cement grout, wood wedges were driven into this space until the voids were almost completely eliminated. In the third step a mining templet was used, cut from a piece of 1-in. by 8-in. lumber which served to check the accuracy of the upper working face without actually lifting the crown rib up to the face. Mining was thus simplified to a considerable extent and made more accurate.

The effect of these improvements on the settlement of the street surface is demonstrated by Fig. 18. The soil conditions were practically the same on both sides of the boundary between contracts S-5 and S-6. Above the north end of contract S-5 the settlement of the street surface amounted to 6 in. and more. By taking advantage of the experiences gained during the construction of S-5 it was possible to reduce the settlement to $2\frac{1}{2}$ or $3\frac{1}{2}$ in. above the adjoining section of S-6.

Although the settlement above contract S-6 was very much smaller than it was above S-5, there was ample evidence, that the pressure of the clay on the temporary support of the clay on S-6 was also smaller. These and similar observations lead to the following conclusions: To obtain the best results both in the tunnel and on the street surface, the clay should be given as little opportunity for deformation as possible. Even if everything has been done to prevent deformation, the deformations are still important enough to mobilize all the strength inherent in the clay. In this respect the undisturbed soft clay behaves like a solid with a high modulus of elasticity. Very small deformations suffice to stress it to the point of incipient failure. Any further deformation reduces the strength of the clay. On several occasions the writer got the impression that the soft clay actually breaks before it becomes plastic, because both grout and air were found to escape from the tunnel through a thick bed of clay toward the surface without evidence for the presence of a

pre-existing communication between the tunnel and the surface through a vein of foreign material.

By reducing the deformation of the clay to the smallest amount compatible with the general method of construction one also reduces (on Chicago tunnels) the danger of breaking the stiff top crust of the soft clay. According to the diagrams of blowout conditions in Fig. 9, the shearing resistance of this crust (stratum a) represents almost 40% of the total shearing force associated with arching above the roof. This crust cannot stand as much deformation as other strata. As soon as it breaks on account of excessive settlement of the crown of the tunnel, the shearing force S decreases instantaneously. In the writer's opinion the remarkably low values of the settlement above contract S-6 were due partly to the stiff top crust remaining intact. This opinion is corroborated by the observation that the crown of the tunnel on S-5 settled not continuously but by jerks which occurred as often as the working face advanced through a distance of about 20 or 25 ft.

Squeeze Measurements as Danger Signals.—The squeeze measurements described herein proved to be very enlightening for both the engineers and the contractors. Therefore it became customary for the contractors to request squeeze measurements whenever the clay in the tunnel seemed to behave abnormally or the settlement of the street surface increased for no apparent reason. Thus the squeeze measurements served unintentionally as a safeguard against accidents.

It has been stated before that the strength of the clay above the tunnels is due largely to the hard upper crust (see Fig. 9). It is further known that the present stage of Lake Michigan was preceded by a low-water stage during which the bed of clay was exposed to surface erosion. The test borings were spaced 300 ft and they can easily miss such a detail as a buried gully. They are also likely to miss isolated pockets of semi-liquid material surrounded by soft clay (see Fig. 5(*a*)). Hence at any stage of construction the working face might have approached an exceptionally soft spot or a buried gully, the existence of which was not known. The first indication of the pending encounter between the tunnel and the bad spot consists in an abnormal increase in the settlement of the street surface, which always induced a call for a squeeze measurement. The characteristics of the normal flow of the clay toward the tunnel were known to the engineers by experience. Hence any abnormal phenomenon such as a flow at an accelerated rate would have induced them to request speedy construction of a bulkhead to be followed by an exploration of the danger zone by means of supplementary test borings. Once the character of a danger zone is known one can modify the method of mining accordingly.

Conclusions

(1) In general, the original program for the test borings and the soil tests was satisfactory. In connection with future investigations of a similar nature in the Chicago district, the following modifications appear desirable. Along open cuts the spacing between drill holes should be reduced from the normal value of about 300 ft to not more than 100 ft. In the danger zones disclosed

by the soil profile, supplementary borings should be made and the stiff top crust of the soft blue clay deserves more attention than it received.

(2) In several places on South State Street the liner-plate method of construction would have led to failure even in case an air pressure of 12 lb per sq in. had been permanently maintained. A blowout would have had catastrophic consequences almost everywhere on this route. Early in 1939, when the contracts were let, it was not yet possible to predict these conditions on the basis of the results of soil tests. Therefore the decision to use the shield method on this route was reached by necessity on the basis of what amounted to a majority vote; opinions were divided. The uncertainties which then existed have been eliminated by developing a method, called the "danger-spot method," which permits a decision on the basis of the results of soil tests as to whether or not the liner-plate method can safely be used on a given contract.

(3) Observations on an experimental section in a liner-plate tunnel have demonstrated the possibility of safely reducing the thickness of the arch and the amount of reinforcement to a fraction of that which is required according to the customary methods of design. The proposed method of design takes advantage of the mechanical properties of soft, undisturbed clay.

(4) The soft Chicago clay in an undisturbed state behaves like a solid material with a high modulus of elasticity. On a tunnel with a width of 20 ft, deformations of a few inches seem to suffice to stress the clay to the point of incipient failure. Hence, by reducing the movement of the clay toward the tunnel to the smallest amount compatible with the method of construction, everything can be gained and nothing is lost. The methods for accomplishing this purpose on the job have been discussed in detail.

(5) Valuable information has been secured by measuring, from time to time, the movement of the clay toward the tunnel and the movement of the members of the temporary support. The test results placed the efforts toward reducing the settlement of the street surface on a rational basis, and the practice of making the test whenever the clay appeared to behave abnormally reduced the risk of accidents due to mining into bad spots whose existence had not been known.

EARTH PRESSURE MEASUREMENTS IN OPEN CUTS, CHICAGO (ILL.) SUBWAY

BY RALPH B. PECK,[7] JUN. AM. SOC. C. E.

SYNOPSIS

Systematic field observations on open-cut portions of the Chicago subway provided a constant check on the loads carried by the bracing as well as the soil movements associated with the excavations. Results of the measurements are given in this paper, together with a comparison of the measured earth pressures with generally accepted theories. It was found that the magnitude of the total lateral pressure was in satisfactory agreement with either the plane or general wedge theories for purely cohesive soils, having no effective internal friction, but that the distribution of the pressure was non-hydrostatic. Measured movements of the sheeting were found to be in accordance with those theoretically necessary for non-hydrostatic distribution. Simple rules are given which are believed to be applicable to the design of bracing for similar cuts in plastic clay deposits.

PART I.—FIELD MEASUREMENTS

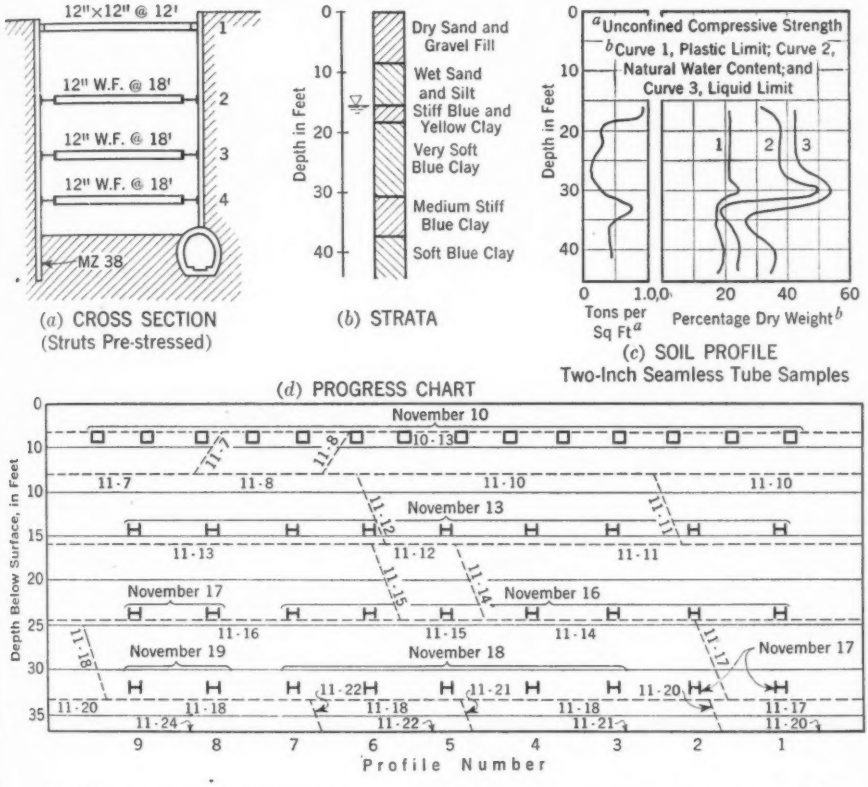
Although the Chicago subway was constructed largely by tunneling methods, a number of special structures were built in open cut. The excavations, frequently of considerable magnitude, were in congested districts near the downtown area in a clay deposit of variable stiffness. It was considered of vital importance to obtain definite information concerning the safety of the bracing of the temporary structures, and to determine the nature and magnitude of the movements of the soil mass and ground surface associated with the excavation. Furthermore, it was considered desirable to obtain quantitative data which could be used as the basis for economical design of the bracing for future excavations. As a result, a systematic program of measurements was undertaken to determine the necessary information.

The open cuts varied in type of bracing, in dimensions, in soil properties, and in method of construction. All these factors were included in the information obtained. The proposed excavations were always outlined by steel piling before any material was removed. Both continuous steel sheeting and H-piles spaced at suitable distances for the insertion of timber lagging were used. All of the excavations were braced with horizontal wales and struts, some of steel and others of wood. The excavations varied in width from 15 ft to 55 ft, in depth from 35 ft to 46 ft, and in length from 100 ft to 1,300 ft. Although the

[7] Asst. Subway Engr., Dept. of Subways and Superhighways, City of Chicago, Chicago, Ill.

construction procedures were all standard practice, each cut was made by a different contractor, with inevitable variations in detail.

The measurements included systematic observations of the settlement of



(e) SUMMARY OF STRUT LOADS, IN KIPS (ESTIMATED ERROR ±15%)

Depth (ft)	Date (Nov. 1939)	Profile (See Fig. (d))								
		9	8	7	6	5	4	3	2	1
4.0	20–27	27	....	33	44	30	....	44	....	....
14.5	20	322	257	270	254	292	220	262	242	226
	27	....	252	249	188	238	....	....	....	....
23.5	20	147	185	161	....	230	164	188	....	140
	27	185	222	199	....	269	218	210	....	157
31.5	20	58	49	40	77	98	81	145	112	150
	27	182	....	252	295	355	248	323	266	262

FIG. 19.—STRUT LOADS IN OPEN CUT, CONTRACT S-3

the adjacent ground surface, of the lateral movements of the sheeting during the excavation process, of the rise of the soil at the bottom of the cuts, and of the settlements of the sheeting itself. In each cut, at least one vertical profile of bracing was selected for determination of the loads carried by the struts.

In all, more than fifty profiles were subjected to measurement, some repeatedly. For steel struts the loads were usually determined by strain gage, and for wooden struts a hydraulic jacking arrangement was used. Hydraulic jacking, when used, was so arranged that errors due to jack friction or to over-jacking

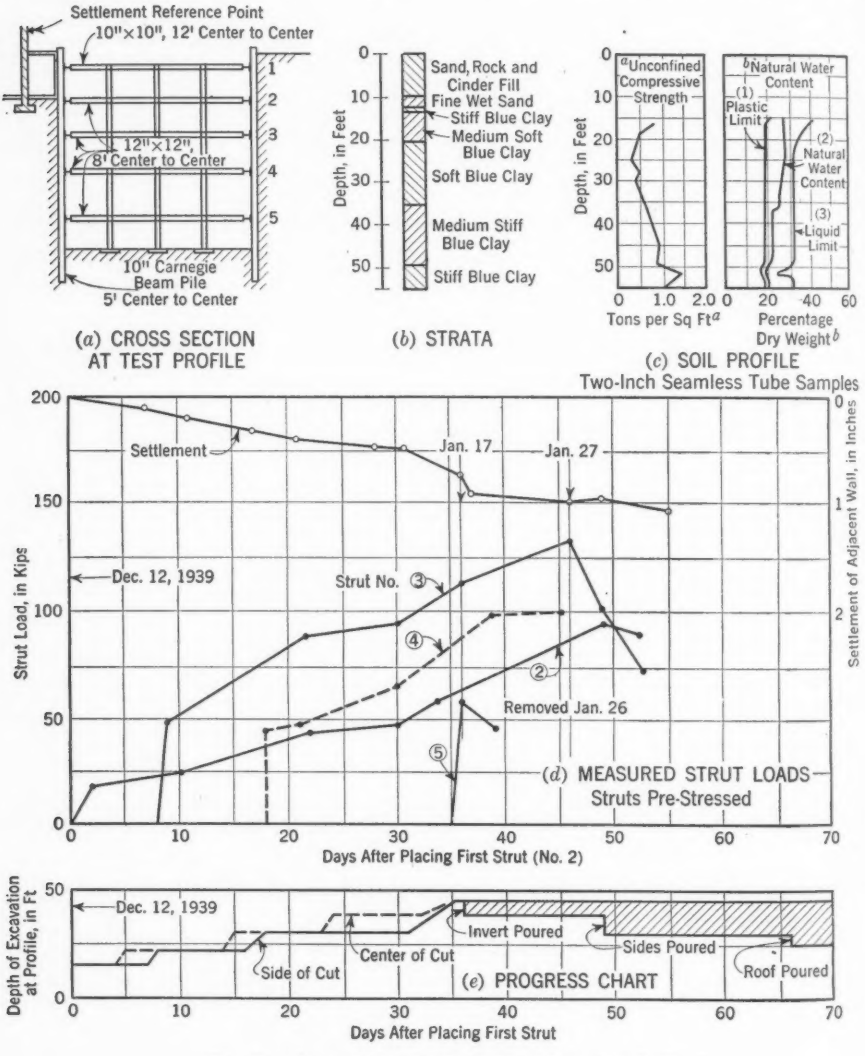


FIG. 20.—STRUT LOADS IN OPEN CUT, CONTRACT S-4B

were eliminated. A detailed discussion of the techniques of measurement and the errors involved was presented elsewhere by the writer.[8] The soil properties adjacent to each cut were determined from the results of laboratory tests on

[8] "The Measurement of Earth Pressures on the Chicago Subway," by Ralph B. Peck, *Bulletin*, A. S. T. M., August, 1941, p. 25.

2-in. diameter samples obtained in seamless tubes from borings taken at 300-ft intervals along the subway right of way.[9] For the comparison of results of the open-cut measurements, the most important physical property of the clay was the unconfined compressive strength, determined on samples 2 in. in diameter and 3.5 in. long at 2.5-ft intervals in the vertical direction. The samples ob-

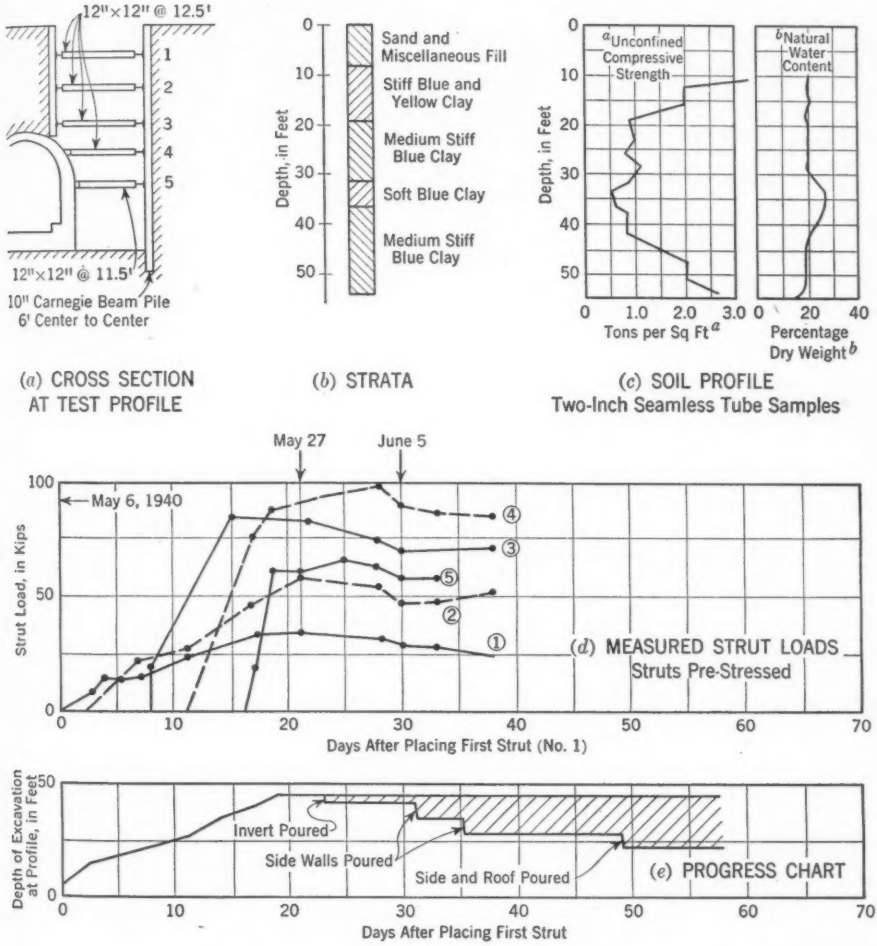


FIG. 21.—STRUT LOADS IN OPEN CUT, CONTRACT S-9C

tained from the borings were disturbed to a certain extent because of the small diameter of the tubes, and an extensive series of tests was conducted to determine the loss in strength during the sampling process. As a result, it was determined that the average unconfined compressive strength of large relatively undisturbed blocks of clay was approximately 1.35 times the value ob-

[9] For details of the soil exploration program, see "Soil Tests Check Chicago Subway Work," by Ralph B. Peck, *Engineering News-Record*, December 7, 1939, p. 87.

tained from the seamless tube samples on the same soil. It was also considered important to determine the order of magnitude of the error in the average unconfined compressive strength for a single boring, introduced because samples were tested only at 2.5-ft intervals. On certain borings, two samples were tested in each 2.5-ft interval and the average value of compressive strength determined from all of the tests. This value was compared with the average

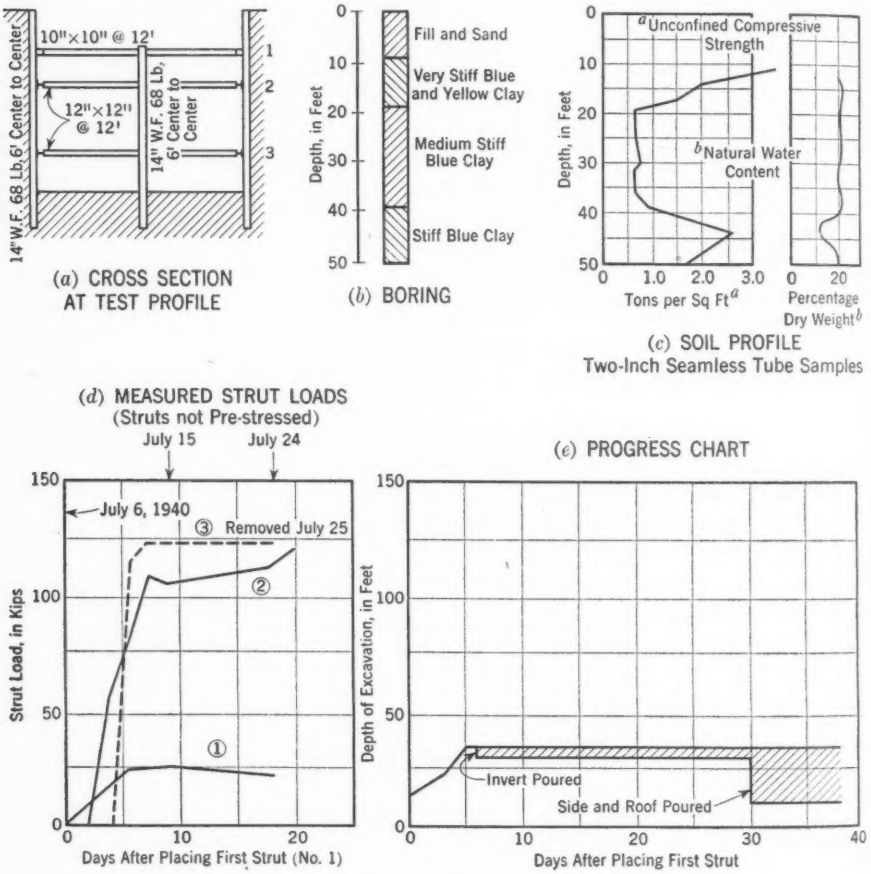
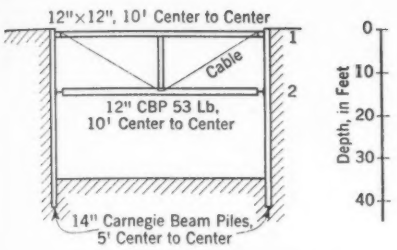


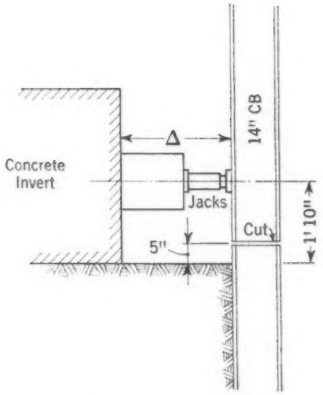
FIG. 22.—STRUT LOADS IN OPEN CUT, CONTRACT S-8A

for alternate samples, representing the usual 2.5-ft spacing. It was found that the difference was about 6%, and that the average compressive strength of the entire boring was probably determined within 10% by the use of the 2.5-ft spacing.

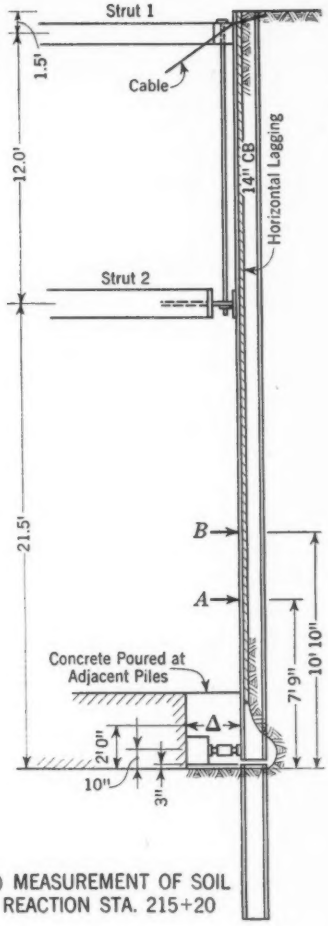
The pertinent information and test results for each open cut are shown in Figs. 19 to 26. In Fig. 19 the progress of the excavation is indicated by the dates along the broken lines. The remaining dates indicate when the struts were placed. In Fig. 19(*c*) the left-hand curve gives the unconfined compressive



(*a*) TYPICAL CROSS SECTION AT TEST PROFILE



(*b*) METHOD FOR OBTAINING SOIL REACTION, STA. 213+85



(*c*) MEASUREMENT OF SOIL REACTION STA. 215+20

(*d*) SOIL PROFILE
Two-Inch Seamless Tube Samples

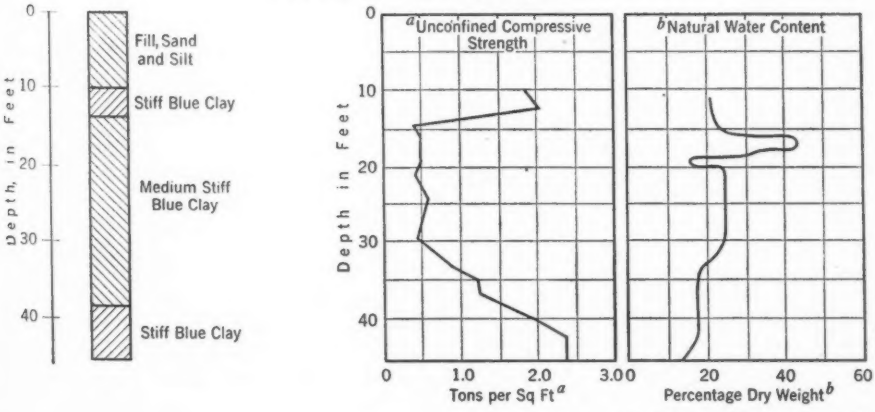


Fig. 23.—continued on page 906.

(*e*) SUMMARY OF TEST RESULTS (*Continued from page 905*)

No.	Description	STATIONS: 213+85	STATIONS: 215+20
1	Load in strut 1 (kips)	9±5	1±2
2	Load in strut 2 (kips)	167±10	137±5
3	Twice the reaction (in kips) at the embedded part of the pile	83±4	82±10
4	Distance of the pile cutoff above the invert (inches)	5	3
5	Measured load in the hydraulic jacks (kips)	41±2	41±5
6	Distance Δ held unchanged at both stations	√	√
7	Subsequent test reduction of load (kips)	8	0
	Result of load reduction (Item 7):		
8	Decrease in Δ (inches)	0.2	0.21
9	Inward movement of point A, Fig. (*c*), as measured on the cut pile (inches)	....	0.4
10	Inward movement of point B, Fig. (*c*), as measured on the adjacent pile (inches)	....	0.0013
11	Excavation completed (1940)	Oct. 8	Oct. 24
12	Invert poured (1940)	....	Nov. 12
13	Measurements made	Oct. 14–18	Nov. 13

FIG. 23.—LATERAL EARTH PRESSURE MEASUREMENTS, CONTRACT D-6E

strength in tons per square feet; and curve 1 is the plastic limit, curve 2 is the natural water content, and curve 3 is the liquid limit. In Figs. 20 to 22, the estimated error in strut loads is approximately 10 kips. Cross sections of each cut are shown in Figs. 19 to 25, together with a description of the soil profile and the results of unconfined compressive strength and natural water-content determinations. Diagrams showing the progress of excavation are shown in Figs. 19 to 22, as well as the measured strut loads corresponding to various excavation stages for contracts S-3, S-4B, S-9C, S-8A, and D-6E (for location, see Fig. 1). Figs. 25 and 26 show the excavation progress, strut-load determinations and soil movements at one profile on contract S-1A, and Table 1 contains the measured strut loads for 21 profiles in contract S-1A, measured soon after excavation reached grade at each profile.

TABLE 1.—STRUT LOADS, CONTRACT S-1A; DEPTH 36.5 FT

Station	Profile	Load (in Kips) in the Strut, at Depths (in Ft) of: 4	13	23	31	Total load, *Q*	Center of pressure,[d] *N*
61+05	3	...[b]	342	...[c]	...[b]	....	....
60+90	4	...[b]	335	...[c]	...[b]	....	....
60+45	7	...[b]	466	...[c]	...[b]	....	....
60+30	8	8.5	...[b]	...[c]	...[b]	....	....
60+15	9	...[b]	...[b]	...[c]	237	....	....
59+85	11	...[b]	338	82	232	652	0.43
59+70	12	3.8	223	316	89	628	0.43
59+25	15	...[b]	286	284	82	652	0.45
59+10	16	...[b]	258	196	88	542	0.46
58+95	17	...[b]	300	254	130	684	0.45
58+80	18	...[b]	271	199	142	612	0.44
58+65	19	...[b]	294	185	214	693	0.42
58+50	20	...[b]	228	254	123	605	0.43
58+35	21	...[b]	244	249	146	639	0.42
58+20	22	...[b]	196	202	147	545	0.41
58+05	23	1.0	249	193	105	547	0.45
57+90	24	...[b]	261	224	83	568	0.51
57+75	25	...[b]	278	146	124	548	0.46
57+60	26[a]	...[b]	170	147	109	426	0.42
57+45	27	...[b]	259	187	148	594	0.43
57+30	28	...[b]	219	166	116	501	0.44
57+15	29	...[b]	189	173	131	493	0.42

[a] At a stair well; wales not blocked to the sheeting. [b] Strut load not measured. [c] No strut at this level. [d] Center of pressure *N* is the ratio of the height of the center of gravity of the measured strut loads above the bottom of the cut, to the total depth of the cut.

The closeness of superimposed loads to the excavation is not indicated in Figs. 19 to 24. Although the lateral pressure produced by the weight of adjacent structures was insignifi-

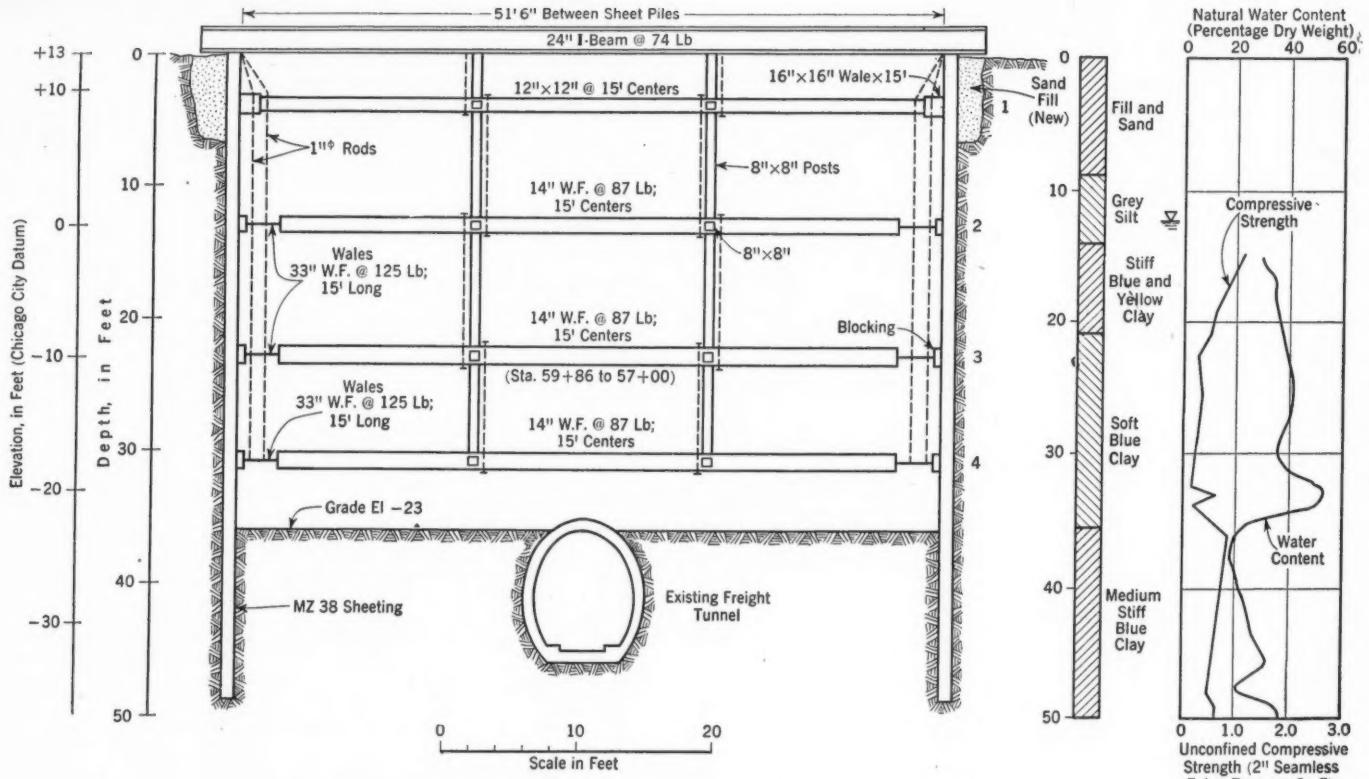
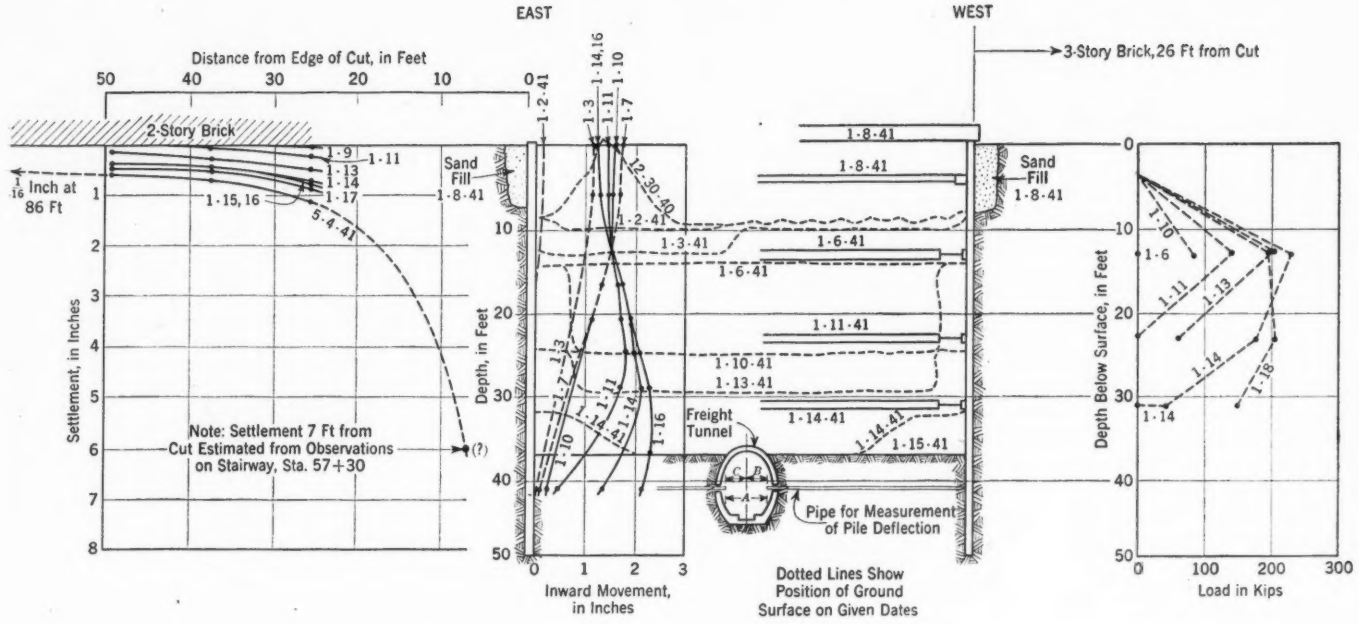


FIG. 24.—OPEN-CUT BRACING AND SOIL PROFILE, CONTRACT S-1A, STATIONS 61+36 TO 57+00 (STRUTS PRE-STRESSED)


EAST
WEST
Distance from Edge of Cut, in Feet
50 40 30 20 10 0
2-Story Brick
3-Story Brick, 26 Ft from Cut
1/16 Inch at 86 Ft
Sand Fill
1·8·41
Settlement, in Inches
Depth, in Feet
Depth Below Surface, in Feet
Note: Settlement 7 Ft from Cut Estimated from Observations on Stairway, Sta. 57+30
Freight Tunnel
Pipe for Measurement of Pile Deflection
Inward Movement, in Inches
Dotted Lines Show Position of Ground Surface on Given Dates
Load in Kips

(*a*) SETTLEMENT OF SURFACE

(*b*) PILE MOVEMENTS AND EXCAVATION PROGRESS

(*c*) STRUT LOADS

FIG. 25.—MEASUREMENTS ON OPEN-CUT PROFILE 22, STATION 58+20, CONTRACT S-1A

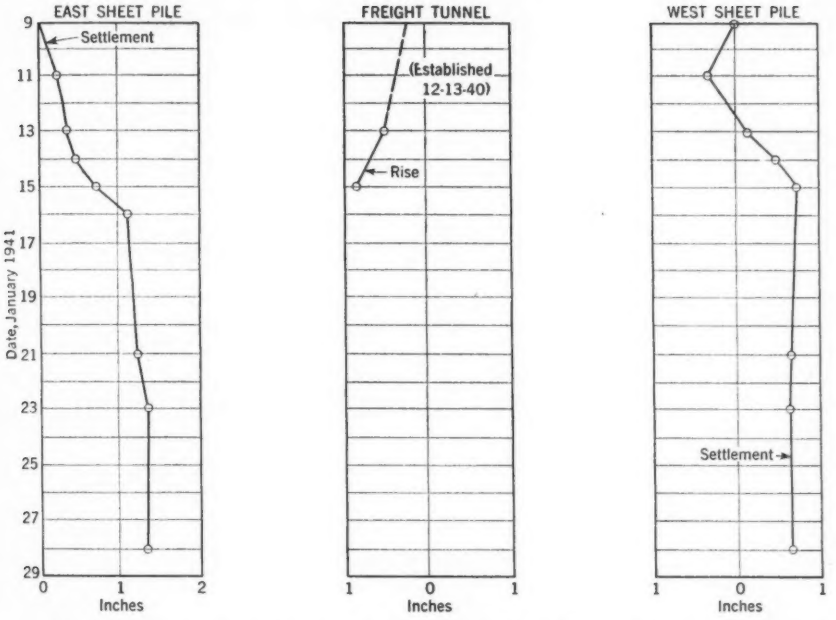


FIG. 26.—SETTLEMENT OF SHEET PILE WALLS, AND RISE OF FREIGHT TUNNEL (See Fig. 25(*b*))

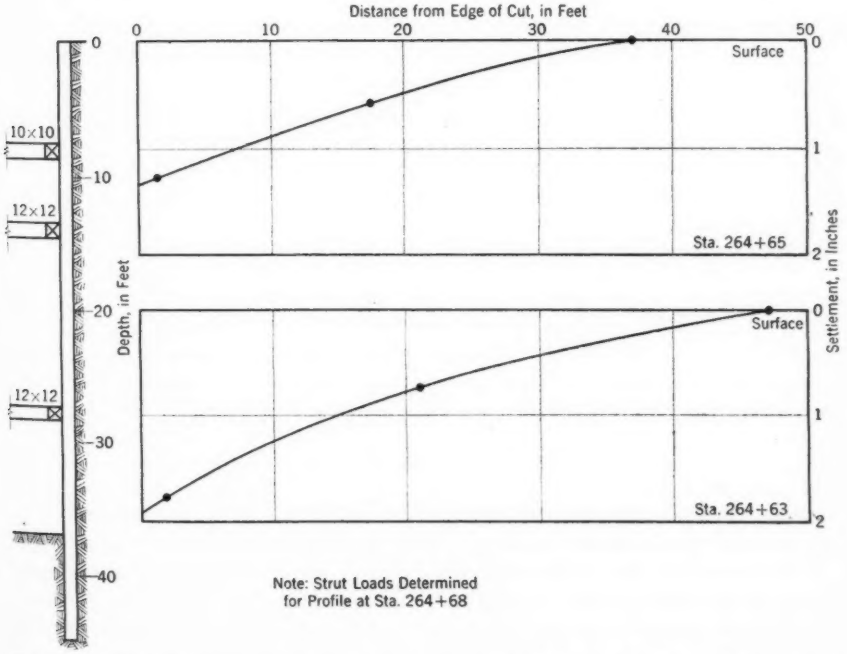


FIG. 27.—SETTLEMENT ADJACENT TO OPEN CUT, CONTRACT S-8A, FOUR MONTHS AFTER COMPLETION

cant compared to the total lateral pressure, the distance from each open cut to the nearest structure is recorded in Table 2.

The presence of the two-story frame building immediately adjacent to the cut on contract S-8A permitted accurate settlement observations very close to the excavation. The results are shown in Fig. 27.

TABLE 2.—DISTANCE FROM EACH OPEN CUT TO THE NEAREST STRUCTURE

Cut (see Fig. 1)	Distance (ft)	Structure	
		Stories	Type
S-3	50	5	Brick[a]
S-4B	9	3	Brick[b]
S-9C	50	3	Brick
S-8A	0	2	Wood frame
D-6E	40	2	Brick
S-1A	26	2	Brick

[a] On 60-ft piles. [b] 7,000 lb per foot of wall.

PART II.—THEORY OF TOTAL LOADS

In order to generalize the results of the earth pressure measurements, it is desirable to determine a theoretical basis upon which the results can be compared. The so-called classical earth pressure theories have been developed for many years to include homogeneous materials whose shearing resistance is made up of both cohesion and internal friction. Although the Chicago soils in which the cuts were made were predominantly plastic clay, the presence of an overlying sand layer required modification of the existing theories. Furthermore, the influence of progressive failure in reducing the average strength of cohesive deposits, and the real angle of internal friction exhibited by large masses of clay in the field, represented unknown factors which could not be determined by laboratory tests. In this section, the theoretical expressions for total lateral earth pressure are modified to include the effect of the sand layer, and are written in such a form as to permit the determination, from the measured pressures and soil properties, of the effect of progressive failure, and of the angle of internal friction which was actually effective.

1. Rankine's Earth Pressure Theory.—In a normally consolidated clay deposit with a horizontal surface, the intensity of vertical stress on a horizontal section is given by the expression

$$\sigma_v = \gamma H_z \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5a)$$

in which γ = density of the overlying soil mass; and H_z = vertical distance from the ground surface to the horizontal section. In an undisturbed soil mass, the intensity of horizontal pressure on a vertical plane is given by the expression

$$\sigma_h = K_0 \gamma H_z \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5b)$$

in which K_0 = coefficient of natural earth pressure, or earth pressure at rest. The value of K_0 for clays in their natural state is not yet known.

If the bed of clay is given an opportunity for lateral expansion to a very great depth until a state of plastic equilibrium is reached, the lateral intensity of pressure decreases to the smallest value compatible with equilibrium. Its

value, known as the active earth pressure, is

$$\sigma_A = \gamma H_z \left[\tan^2 \left(45° - \frac{\phi}{2} \right) - \frac{2c}{\gamma H_z} \tan \left(45° - \frac{\phi}{2} \right) \right] \ldots\ldots (6)$$

in which ϕ = effective angle of internal friction of the clay; and c = cohesion of the clay. The total lateral pressure from the surface to depth $H_z = H$ is equal to

$$P_A = \int_0^H \sigma_A \, dH_z = \tfrac{1}{2} \gamma H^2 \tan^2 \left(45° - \frac{\phi}{2} \right) - 2c H \tan \left(45° - \frac{\phi}{2} \right) \ldots (7)$$

Eqs. 6 and 7 were probably first derived by Résal (1910) as an extension of Rankine's theory. A derivation in English was given by the late William Cain, M. Am. Soc. C. E., in 1916.[10] Eq. 7 may be expressed in another useful form: The unconfined compressive strength of a clay sample, q, may be expressed in terms of the cohesion and effective angle of internal friction by the equation

$$q = \frac{2c}{\tan \left(45° - \frac{\phi}{2} \right)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Substituting Eq. 8 in Eq. 7, the expression

$$P_A = \tfrac{1}{2} \gamma H^2 \left(1 - \frac{2q}{\gamma H} \right) \tan^2 \left(45° - \frac{\phi}{2} \right) \ldots\ldots\ldots\ldots (9)$$

is obtained.

The resultant earth pressure given by Eqs. 7 and 9 is horizontal, and its point of application is below the lower third-point. For $H = \frac{2q}{\gamma}$, it is at infinite depth below the bottom of the cut. The magnitude of the resultant earth pressure is often compared with that of a fluid of equal unit weight, which exerts a total lateral pressure

$$P_F = \tfrac{1}{2} \gamma H^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

The ratio of P_A to P_F is known as the hydrostatic pressure ratio for active earth pressure, and is denoted by K_A. Its value is given by either of the equations

$$K_A = \tan^2 \left(45° - \frac{\phi}{2} \right) - \frac{4c}{\gamma H} \tan \left(45° - \frac{\phi}{2} \right) \ldots\ldots\ldots (11a)$$

or

$$K_A = \left(1 - \frac{2q}{\gamma H} \right) \tan^2 \left(45° - \frac{\phi}{2} \right) \ldots\ldots\ldots\ldots (11b)$$

In actual experience, opportunity for lateral expansion sufficient to develop a state of plastic equilibrium throughout the soil mass is never realized. Such a movement would result in the full development of shearing resistance on two

[10] "Earth Pressure, Retaining Walls and Bins," by William Cain, New York, N. Y., 1916, p. 186.

intersecting sets of sliding planes, each of which makes an angle of $45° - \frac{\phi}{2}$ with the vertical as indicated in Fig. 28. Actually the expansion is restricted to a wedge-shaped zone extending behind the wall supporting the earth mass, and having a depth limited approximately to the depth H of the wall. If sufficient movement of the wall is permitted to develop the shearing resistance along the plane which intersects the wall at depth H, the validity of Eqs. 11 is not impaired, provided that two other conditions are also fulfilled. A bond

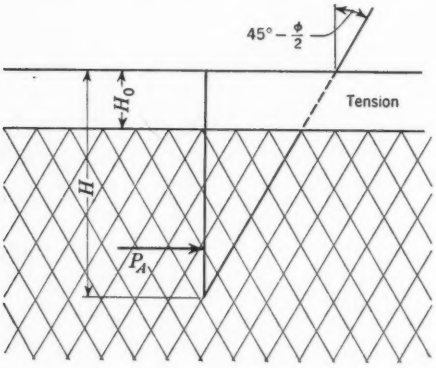


FIG. 28

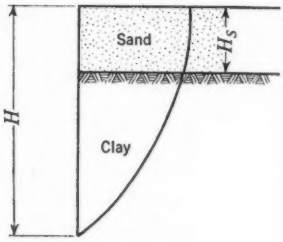


FIG. 29

must exist between the lateral support and the clay, capable of resisting tension to a depth, according to Eq. 6, of

$$H_0 = \frac{q}{\gamma} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

and the deformation conditions must be such that the point of application of the resultant pressure is identical with the theoretical location. In an actual open cut, these latter two conditions are never satisfied, because bracing is always inserted near the ground surface, and lateral expansion is greatly restricted. The zone of tension in the upper part of the deposit no longer can exist, the center of pressure rises, and Eq. 6 is no longer applicable. The sliding surface becomes curved, and the resultant earth pressure is not horizontal. Ignorance of these factors can be expected to introduce errors into the computed values of the total pressure. The theory is adequate to serve, nevertheless, as the starting point in the analysis of the experiment results.

Eqs. 11 refer strictly to a homogeneous clay mass possessing everywhere an unconfined compressive strength q and an angle of internal friction ϕ. Natural clay deposits are not homogeneous, and the unconfined compressive strength is variable. To use equations containing the quantity q, it is then necessary to substitute in place of it the arithmetic average of the compressive strength of the clay, denoted by q_a. In the case of the Chicago clays, another complication is introduced by the presence of a deposit of cohesionless material in the upper portion of the soil mass. For computation, it is convenient to

transform the cohesionless layer into an equivalent layer of clay possessing the same shearing resistance along the potential sliding surface as the existing sand, but having an angle of internal friction equal to that of the actual clay deposit.

The shearing resistance of the sand along the portion of the sliding wedge located above the clay is equal to

$$S_s = \sigma \tan \phi_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

in which: σ = the average normal stress on the sand portion of the sliding surface; and ϕ_1 = angle of internal friction of the sand. The average normal stress (neglecting the inclination of the shearing surface which actually is almost vertical, as indicated in Fig. 29) is equal approximately to

$$\sigma = K_s \times \tfrac{1}{2}\, \gamma_1 H_s \ldots\ldots\ldots\ldots\ldots\ldots\ldots (14a)$$

and the average shearing resistance

$$S_s = K_s \times \tfrac{1}{2}\, \gamma_1 H_s \tan \phi_1 \ldots\ldots\ldots\ldots\ldots\ldots (14b)$$

In Eqs. 14: K_s = hydrostatic pressure ratio for the cohesionless material; γ_1 = density of the cohesionless material; and H_s = depth of cohesionless layer. The shearing resistance of the substitute clay layer must also be equal to S_s and is given by the expression

$$S_s = c_s + \sigma \tan \phi \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

in which: c_s = cohesion of the substitute clay layer; and ϕ = angle of internal friction of the clay deposit. Substituting Eq. 14*b* for S_s in Eq. 15 and solving for the cohesion, it is found that

$$c_s = \tfrac{1}{2}\, K_s\, \gamma_1\, H_s\, (\tan \phi_1 - \tan \phi) \ldots\ldots\ldots\ldots\ldots (16a)$$

The unconfined compressive strength of the substitute clay layer would then be, from Eq. 8,

$$q_s = \frac{2\,c}{\tan\left(45° - \dfrac{\phi}{2}\right)} = \frac{1}{\tan\left(45° - \dfrac{\phi}{2}\right)} \times K_s\, \gamma_1\, H_s\, (\tan \phi_1 - \tan \phi) \ldots (16b)$$

The average unconfined compressive strength of the entire deposit from the surface to depth H would become

$$q_a = \frac{1}{H}\left[\frac{K_s\, \gamma_1\, H^2_s}{\tan\left(45° - \dfrac{\phi}{2}\right)} (\tan \phi_1 - \tan \phi) + (H - H_s)\, q_c\right] \ldots\ldots (17)$$

Eqs. 16*a* and 17 contain the quantity K_s, which can only be estimated, by a procedure to be described subsequently. They afford only estimates of the influence of the sand layer, rather than exact evaluations.

One further soil property must be considered in the adaptation of the theory to an actual soil mass. The value of the average unconfined compressive strength used in the computations is based upon tests of individual samples as

representative of the clay deposit as possible. For each sample, the maximum strength is recorded as the unconfined compressive strength. For most natural clays, the maximum strength is attained at relatively small strain, and the compressive strength decreases rapidly to a constant value which persists throughout the additional strain, as shown in Fig. 30. Since some of the layers in a natural deposit are stiffer than others, at a given deformation the clay at certain points along the sliding surface may have attained its maximum resistance and experienced the decrease in strength, whereas at other points the maximum may not have been reached. As a result, the effective shearing resistance along the surface is always less than the average computed from the maximum strengths of individual samples. When sufficient deformation has taken place to reach the constant portion of the stress-strain curve for each component of the soil, it is apparent that the average resistance is only a fraction of that determined from the tests. The ratio between the shearing resistance actually developed along the sliding surface, and that determined by averaging the separate shearing resistances of the individual samples is denoted by n. Eq. 17 can be modified to take account of this phenomenon (known as progressive failure) by replacing q_c by $n\,q'_c$, in which q'_c is the average unconfined compressive strength of practically undisturbed clay samples as determined in the laboratory. No reduction is experienced in the sand layer because the stress-strain curve for loose sands, as exist over the Chicago clay deposits, does not exhibit a peak before attaining a constant shearing resistance as deformation proceeds. The final form of Eq. 17 is therefore

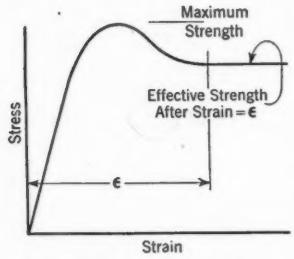


FIG. 30.—STRESS-STRAIN CURVE

$$q_a = \frac{1}{H}\left[\frac{K_s\,\gamma_1\,H^2_s}{\tan\left(45^\circ - \frac{\phi}{2}\right)}(\tan\phi_1 - \tan\phi) + (H - H_s)\,n\,q'_c\right] \quad \ldots(18a)$$

and the theoretical relation Eq. 11*b* becomes

$$K_A = \left(1 - \frac{2\,q_a}{\gamma_a\,H}\right)\tan^2\left(45^\circ - \frac{\phi}{2}\right) \quad \ldots(18b)$$

in which the average density, γ_a, is determined by the equation

$$\gamma_a = \frac{1}{H}\left[H_s\,\gamma_1 + (H - H_s)\,\gamma_c\right] \quad \ldots(19)$$

γ_c being the average density of the clay.

It is important to note that Eq. 18*b* represents a linear relationship between K_A and $\frac{2\,q_a}{\gamma_a\,H}$, as shown in Fig. 31. Furthermore, the intercept of the straight line on the K_A-axis must be equal to $\tan^2\left(45^\circ - \frac{\phi}{2}\right)$, and the intercept on the

$\frac{2\,q_a}{\gamma_a H}$-axis must be equal to unity. These facts are of the greatest importance, because they enable the values of ϕ and n for the clay deposit to be determined from the experimental values of earth pressure. The procedure is described in Part III.

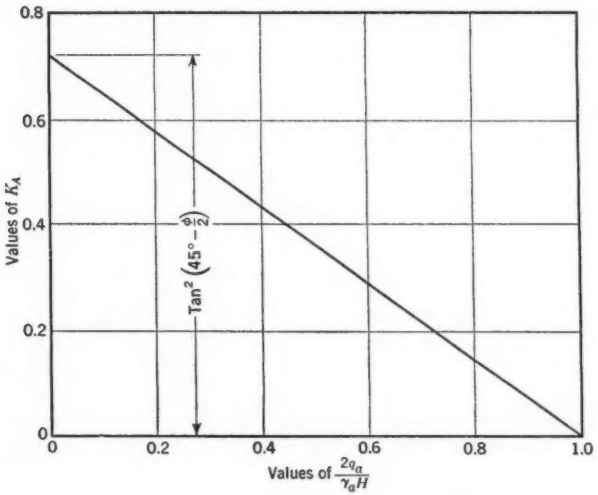


FIG. 31

2. General Wedge Theory for Earth Pressures.—When an open cut is braced with struts, the deformation at the top of the cut is greatly restricted and the conditions for tensile stresses in the upper part of the soil mass are no longer satisfied. Pressure exists everywhere within the wedge, and the center of pressure is correspondingly raised. The sliding surface is no longer plane, but takes the shape of a curve which intersects the surface at a right angle. The resultant earth pressure is not horizontal, in general, but is inclined to the horizontal at an angle θ.

The total pressure against the sheeting under these circumstances may be computed by consideration of the equilibrium of the failure wedge, under the assumption that the shape of the curved boundary is that of a logarithmic spiral. This curve was chosen for its close approach to observed conditions and its simplicity, because the resultant of the normal and shearing forces on the curved surface passes through the center of the spiral and may be eliminated from computations if this point is taken as the center of moments. For the special case when $\phi = 0$, the logarithmic spiral becomes a circle with its center on a horizontal line at the elevation of the ground surface. The forces considered are shown in Fig. 32. They include W, the weight of the wedge; P_A, the resultant earth pressure; S, the total shearing resistance along the curved sliding surface; and F, the resultant normal force on the curved surface. Denoting the lever arms of these forces with respect to the center of rotation

O by the symbol l, the value of P_A is found to be

$$P_A = \frac{W\, l_w - S\, l_c}{l_r} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

The total shearing resistance S is equal to the product of one half the average unconfined compressive strength and the length L of the curved surface. The weight W acts at the center of gravity of the wedge. Several positions are

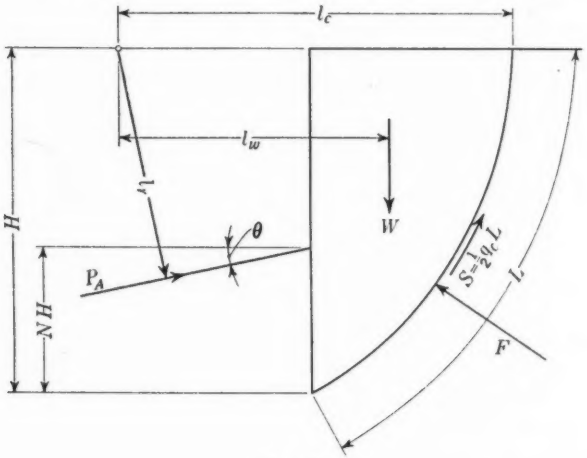


FIG. 32

assumed for the center of rotation along the horizontal line, and the maximum value of P_A is taken as the active earth pressure. The value of the hydrostatic pressure ratio is defined as the ratio of the horizontal component of P_A to the lateral pressure exerted by the fluid of equal density. Thus

$$K_A = \frac{P_A \cos\theta}{\frac{1}{2}\gamma_a H^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

Values of K_A have been computed for different values of the inclination of the resultant earth pressure, for different positions of the center of pressure, and for different values of cohesion. The results are shown in graphical form in Fig. 33. Each of these charts pertains to a particular value of the inclination of the resultant pressure. Values of K_A may be determined for any location of the center of pressure, and for any value of cohesion, expressed in terms of the quantity $\frac{H\,\gamma}{2\,q}$. It should be recalled that Fig. 33 is constructed on the assumption that $\phi = 0°$. The values of K_A determined by Rankine's theory are also shown in Fig. 33. They appear as horizontal lines, inasmuch as the values are independent of the location of the center of pressure. For either of the values of θ it is observed that a particular value of the position of the center of pressure gives results by the general wedge theory which are practically identical to those given by Rankine's theory. For values of $\theta = 0°$, and 20°, the re-

spective centers of pressure for this condition are approximately at 0.37 H and 0.45 H. Similar charts have also been prepared for other values of ϕ, in which case the cohesion of the clay is determined by Eq. 8. However, the

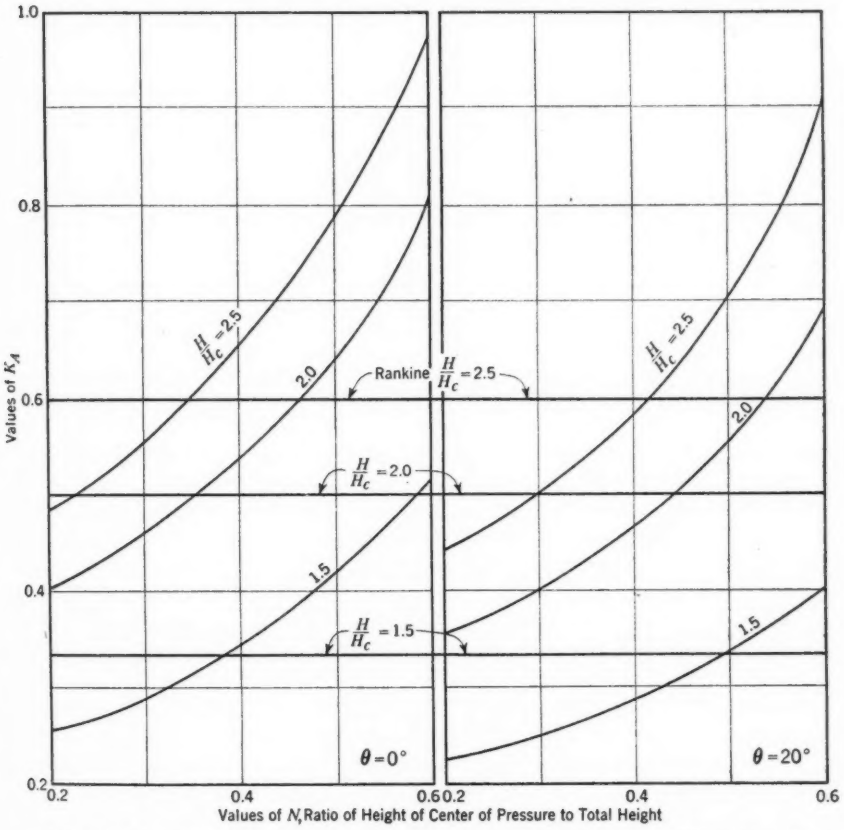


FIG. 33.—RESULTS OF EARTH PRESSURE COMPUTATIONS; GENERAL WEDGE THEORY (ϕ = 0°)

agreement with the field data is more satisfactory in the case of Fig. 33, and the other charts are not shown.

PART III.—COMPARISON OF MEASURED AND THEORETICAL TOTAL LOADS

The essential data regarding the open cuts are shown in Figs. 19 to 27 and Table 1. The results are summarized in tabular form in Table 3. In Table 3(a), the symbol H indicates the total depth of the cut; H_s the thickness of the overlying sand layer, q'_c the average unconfined compressive strength of practically undisturbed clay samples between the clay surface and the bottom of the cut; K_s the hydrostatic pressure ratio for the sand; P_l the lateral pressure of a fluid having a density equal to γ_c; and L_p the horizontal distance between vertical profiles of struts. In Table 3(b), the symbols Q_1, Q_2 . . . indicate the

TABLE 3.—COMPARISON OF MEASURED AND

Cut (see Fig. 1)	Station	(a) TOTAL LATERAL PRESSURE												
		H	H_s	$H - H_s$	q'_c	K_s	γ_c	γ_a	P_l	Q	L_p	P_A	K_A	$\frac{2q_c}{\gamma_a H}$
		(1)[a]	(2)[a]	(3)[a]	(4)[b]	(5)	(6)[c]	(7)[c]	(8)[d]	(9)[d]	(10)[a]	(11)[e]	(12)	(13)
S9C	....	44	9	35	3,280	1.0	130	126	124	331	12.5	26.5	0.214	0.748
S8A	....	37	9	28	2,950	1.0	130	125	86	258	12	21.5	0.250	0.788
D6E	213+85	35	8	27	2,270	1.0	126	123	75	259	10	25.9	0.346	0.664
D6E	215+20	35	8	27	2,270	1.0	126	123	75	220	10	22.0	0.293	0.664
S4B	....	23	12	11	2,050	0.5	122	116	31	73	8	9.13	0.295	0.701
S4B	....	30	12	18	1,730	0.5	124	118	53	181	8	22.7	0.428	0.526
S3	Average	33	16	17	1,170	1.0	114	112	61	558	18	31.0	0.508	0.511
S3	Average	39	16	23	1,310	1.0	118	115	87	750	18	41.7	0.478	0.443
S1A	Average	36.5	15	21.5	1,320	1.0	118	115	76	626	15	41.7	0.547	0.464
S1A	Average	36.5	15	21.5	1,320	1.0	118	115	76	586	15	39.1	0.515	0.464
S1A	58+21	25	15	10	1,470	1.0	114	112	35	140	15	9.33	0.267	0.722
S1A	58+21	27	15	12	1,420	1.0	114	112	41	264	15	17.6	0.429	0.661
S1A	58+21	34.5	15	19.5	1,270	1.0	117	114	68	448	15	29.9	0.433	0.484
S1A	58+21	36.5	15	21.5	1,320	1.0	118	115	76	550	15	36.6	0.482	0.464

Units of Table (a) are: [a] feet; [b] pounds per square foot; [c] pounds per cubic foot; [d] kips = 1,000 lb; [e] kips per foot

measured load in individual struts, and H_1, H_2 . . . the vertical distance between the surface and the strut.

Table 3 also contains the total measured lateral pressure Q per lineal foot of the cut, and the ratio N defined as the distance from the point of application of the lateral pressure to the bottom of the cut divided by the total depth of the cut. To determine these quantities, it is necessary to know the bottom reaction, or the shear in the vertical members of the sheeting at depth H. Whether or not the value of this quantity is important enough to warrant inclusion in the total load Q depends upon the nature of the different cuts. In contract S-1A (see Fig. 1) the greatest deflection of the sheeting occurred at the bottom of the excavation. This type of deflection corresponds to a pressure distribution having small values at the bottom, and the bottom reaction is likely to be small.[11, 12, 13] In contract S-3, the sheeting on one side rested on top of an existing freight tunnel of unreinforced concrete which was capable of supporting an inconsequential lateral load after exposure. Similarly in contract S-4B the results of the strut loads indicate plainly that the intensity of lateral pressure decreased from the midheight to the bottom of the cut and that the major part of this pressure has been taken by the struts. On the contrary, the vertical H-piles on contract D-6E penetrated a stiff layer below the bottom of the cut, and the large clearance between the lower strut and the bottom of the cut induced a deflection curve with a maximum value at some distance above the bottom of the cut. As a result, the bottom reaction was considered likely to be of considerable importance, and it was measured by the

[11] "Distribution of the Lateral Pressure of Sand on the Timbering of Cuts," by Karl Terzaghi, *Proceedings*, International Conference on Soil Mechanics, Vol. I, Cambridge, Mass., 1936.

[12] "A Fundamental Fallacy in Earth Pressure Computations," by Karl Terzaghi, *Journal*, Boston Soc. of Civ. Engrs., April, 1936, p. 78.

[13] "General Wedge Theory of Earth Pressure," by Karl Terzaghi, *Transactions*, Am. Soc. C. E., Vol. 106, 1941, pp. 68, 94.

THEORETICAL TOTAL LOADS IN OPEN CUTS

(*b*) INDIVIDUAL STRUT LOADS

H	Strut 1		Strut 2		Strut 3		Strut 4		Strut 5		*N*	*Q*
	H_1	Q_1	H_2	Q_2	H_3	Q_3	H_4	Q_4	H_5	Q_5		
(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)
44	5.4	35	12.6	58	19.5	82	25.8	91	33.0	65	0.52	331
37	7.8	25	12.2	109	28.1	124	....	....	....	....	0.48	258
35	5.5	9	13.6	167	33.2	(83)[/]	....	....	....	....	0.44	259
35	1.5	1	13.5	137	34	(82)[/]	....	....	....	....	0.40	220
23	0	0	11	25	18.6	48	....	....	....	....	0.30	73
30	0	0	11	43	18.6	88	27.4	50	....	....	0.36	181
33	4	36	14.5	260	23.5	173	31.5	89	....	....	0.42	558
39	4	36	14.5	232	23.5	209	31.5	273	....	....	0.42	750
36.5	4	8	13	381	....	....	31	237	....	....	0.46	626
36.5	4	2	13	251	23	203	31	130	....	....	0.44	586
25	4	0	13	140	....	....	....	....	....	....	0.48	140
27	4	0	13	205	23	59	....	....	....	....	0.45	264
34.5	4	0	13	230	23	178	31	40	....	....	0.46	448
36.5	4	0	13	196	23	204	31	150	....	....	0.40	550

[/] Values in parentheses obtained by burning H-pile free at bottom of cut.

method indicated in Fig. 23. For cuts S-8A and S-9C, it was concluded that the bottom reaction was not important, although it is possible that in S-9C a small reaction might have been required to produce the state of stress measured in the bracing. As a result, the cut on contract D-6E was the only one in which the bottom reaction was taken into consideration in the computations.

To compute the real value of K_A, the average density of the soil throughout the height H must be determined by Eq. 19. For this computation, the density of the sand above the clay surface was estimated at 110 lb per cu ft. The density of the clay was computed from the average natural water content and a specific gravity of the solid matter assumed equal to 2.74. The values of γ_c and γ_a are given in Table 3(*a*).

The theoretical equation (Eq. 18*a*) for the average equivalent unconfined compressive strength of the deposit contains two values, K_s and tan ϕ_1, which can only be estimated. Since the sand above the clay is usually loose and somewhat silty, it is likely that ϕ_1 varies between the limits of 30° and 35°. It is most likely closer to 30°, but both values have been used in subsequent calculations. The value of K_s depends upon the lateral strain that develops in the sand during excavation. It must be selected for each cut individually, because of its dependence upon the method of excavation and bracing. As an example of the method of evaluation, cut S-9C is used. In the vicinity of the cut, the depth of sand was 9 ft. For $K_s = 1$, the earth pressure exerted by the sand is $\frac{1}{2} \times 110 \times 9^2 = 4{,}450$ lb per ft = 4.45 kips per ft. The top row of struts was located at a depth of 5.4 ft, and the next at 12.6 ft below the surface. Thus the top row received almost the full lateral pressure of the sand. The greatest load in this strut was 35 kips, or 2.80 kips per ft, corresponding to a hydrostatic pressure ratio $K_s = 0.63$. However, while excavation proceeds, the vertical H-beams pivot about the second row of struts, and by virtue of

the stiffness of the H-beams the strut load is relieved. As a result, the pressure actually exerted by the sand is greater than that indicated by the strut load, and a value of $K_s = 1.0$ was estimated. Conditions were similar for all other cuts except contract S-4B, where the top strut was inoperative and the pressure of the sand was entirely carried by bending in the upper part of the H-beams. Therefore, much greater opportunity for lateral expansion of the sand existed in S-4B than in the other cuts, and a value of $K_s = 0.5$ was estimated correspondingly. An error of 25% in the estimated values of K_s would result in an error in the computed shearing resistance of less than 2% on contracts S-9C, S-8A, and S-4B, less than 10% on D-6E, and approximately 15% on S-1A and S-3.

In Table 3, the results of measurements on contract S-4B after the cut exceeded a depth of 30 ft are not included because measurements were not made on all profiles simultaneously, and certain strut loads would have to be determined by interpolation. This operation is not considered reliable enough in comparison with the other data. On cuts subsequently measured, care was taken to obtain readings on all struts on the same day when possible.

Table 3(*a*), therefore, contains the experimental values of K_A, and all of the quantities necessary to compute the value of q_a in Eq. 18(*a*), except the unknown properties of the clay, ϕ and n. The most probable values of ϕ and n are those which produce the most satisfactory agreement with the theoretical straight-line relationship shown in Fig. 31, provided a straight-line relationship is found to exist. In Figs. 34(*a*), 34(*b*), and 34(*c*), the experimental values of K_A are plotted against values of $\frac{2\,q_a}{\gamma_a H}$, in which q_a has been computed by Eq. 18*a* on the assumption that $\phi = 17°$. Values of n equal to 0.50, 0.75, and 1.0 were also assumed. A straight-line relationship is seen to exist, indicating that the reasoning leading to Eq. 18*b* is satisfactory. However, in none of these cases do the plotted test results indicate a satisfactory agreement with the theoretical relationship for $\phi = 17°$, shown by the heavy line. It appears evident that the value of ϕ must be appreciably less than 17°. In Figs. 34(*d*), 34(*e*), and 34(*f*), a similar series of plots has been made, in which q_a was determined on the assumption that $\phi = 0°$. In this case, it is observed that the experimental data are in excellent agreement with the theory when $n = 0.75$. It was found by similar computations that no other combination of ϕ and n besides $\phi = 0°$ and $n = 0.75$ gave a reasonable fit to the theoretical relationship. In the charts of Fig. 34, the value of ϕ_1 was taken as 30°. Repetition of the computations with a value of $\phi_1 = 35°$ led to almost identical results. The solution of Eq. 18*b*, therefore, results in the conclusions that, for all practical purposes, in the Chicago subway cuts, the value of ϕ for the clay is equal to 0°, and the factor $n = 0.75$.

The correctness of the values for ϕ and n is influenced by the fact that the upper end of the line is largely determined by points representing contracts S-3 and S-1A, in which an error in the value of K_s has an important effect. Nevertheless, if the position of the line is determined from only those profiles in which a 25% error in K_s involves a negligible error in the result, the value of ϕ cannot possibly exceed 3°, and the value of n is not significantly altered. It

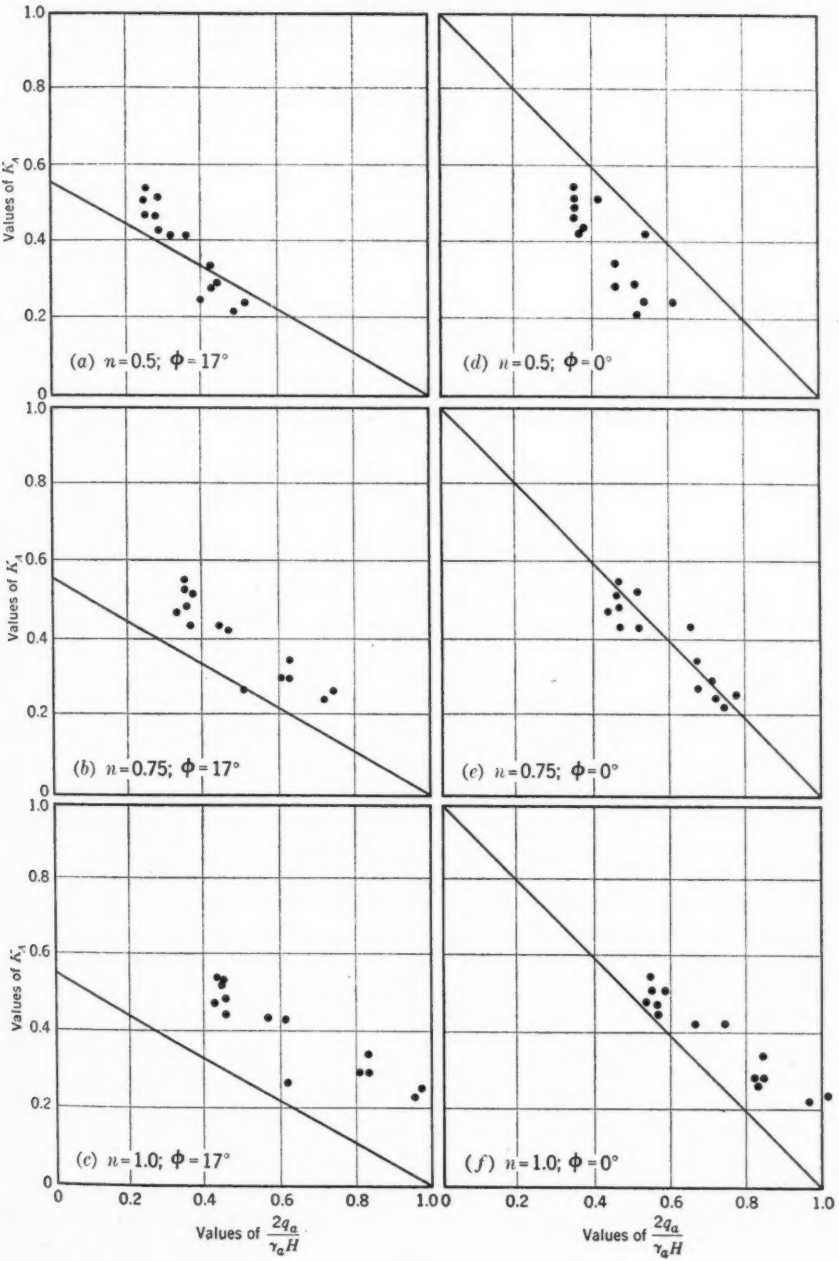

(a) n=0.5; ϕ=17°
(d) n=0.5; ϕ=0°
(b) n=0.75; ϕ=17°
(e) n=0.75; ϕ=0°
(c) n=1.0; ϕ=17°
(f) n=1.0; ϕ=0°
Values of K_A
Values of $\frac{2q_a}{\gamma_a H}$
0
0.2
0.4
0.6
0.8
1.0

FIG. 34

is apparently safe to conclude, therefore, that the mass of clay involved in the deformation accompanying the construction of the Chicago cuts behaved as a purely cohesive mass, without internal friction.

The incomplete development of the shearing resistance, as evidenced by the value $n = 0.75$, might be considered due to the fact that insufficient lateral expansion had taken place to develop the full resistance, and that therefore the measured earth pressure was not the smallest possible value (active earth pressure), but some value between active earth pressure and earth pressure at rest. This interpretation of n, instead of progressive failure, is considered unlikely. The cuts upon which measurements were made were built by different contractors using different systems of lateral support. As long as the shearing resistance of the clay is incompletely developed, a slight change in the amount of lateral yield should have a marked effect on the value of the earth pressure. Nevertheless, the points in Fig. 34 which represent the hydrostatic pressure ratio are all located close to a straight line, indicating that the ratio between active and maximum shearing resistance was consistently equal to 0.75. This fact is in sharp disagreement with the insufficient expansion hypothesis. It appears therefore that the amount of inevitable deformation associated with the construction of such structures is sufficient to reduce the earth pressure to its smallest value, and therefore every effort should be made to keep the lateral yield, which results in settlement, as low as possible.

A study of Table 1, for profiles 15–29 in contract S-1A, indicates the variation from the mean value of the total lateral pressure. These fifteen consecutive profiles were identical except for the ordinary variations in construction procedure. The greatest deviation from the mean was approximately $\pm$ 20%. The physical causes of this scattering are probably numerous, but in any case, provision in design should be made to take it into account.

It is of interest and importance to compare the measured results of the total lateral pressure with the values computed by the general wedge theory, which takes into account the curvature of the sliding surface, the location of the center of pressure, and the inclination of the resultant earth pressure. Inasmuch as no direct measurements were made of the vertical load carried by the sheeting, the value of θ in the experiments in unknown, and only a general comparison is possible. It is observed that the average center of pressure for all the measurements was at $0.43\,H$. This value, according to the curves of Fig. 33, gives results by the general wedge theory when $\theta = 20°$ which are on the average nearly equal to those given by Rankine's theory. It should be emphasized, however, that the center of pressure in Rankine's theory can never be as high as the lower third point, and for small values of $\frac{H\gamma}{2q}$ may even be below the bottom of the cut. The fact that the total lateral pressure is approximately equal to the Rankine value, even when the distribution of the pressure is entirely different, is not altogether surprising after a study of Fig. 33, from which it can be ascertained that the effect of the curvature of the sliding surface is to increase the total lateral pressure, whereas the effect of the vertical component of the pressure is to reduce it. Actually, the observed total pressures

were found to fit the general wedge theory for $\theta = 20°$ slightly better than the plane wedge theory, represented by Eq. 18*b*.

PART IV.—DISTRIBUTION OF PRESSURE

It has been mentioned in Part II that if the shearing resistance is fully developed along the sliding surface forming one of the boundaries of the failure wedge, the total earth pressure can be computed by considering the equilibrium of the wedge. This procedure, however, does not produce any information concerning the distribution of the lateral pressure. The position of the center of pressure depends upon the deformation conditions within the wedge itself. Professor Terzaghi[14] has stated the conditions necessary for various types of pressure distributions. He has shown that if the supporting wall does not yield in a manner approaching rotation about the base of the cut, through a sufficiently large angle to reduce all of the material in the wedge to a state of plastic equilibrium, then the center of pressure will be higher than that predicted by the theory of plasticity. In particular, if yielding is restricted at the top of the cut by the presence of struts, the center of pressure may be expected to rise.

Theoretical considerations do not permit the calculation of the location of the center of pressure, or of the loads carried by individual struts, because the deformation conditions are a function of the manner in which the excavation and bracing are performed. Inspection of Fig. 25, showing the measured deformations accompanying the excavation of one Chicago subway cut, demonstrates that the type of yielding is of the nature to produce a high center of pressure, but quantitative information about the distribution can only be obtained from measurements in the field. An important fact, however, is disclosed by Fig. 25. Almost all of the deformation at a given elevation occurs before excavation reaches that depth. This yielding obviously cannot be prevented entirely by the use of struts. This observation leads to the conclusion that minor variations in construction procedure cannot greatly influence the nature of the deformation, and thus may not have a pronounced effect on the location of the center of pressure. It also leads to the conclusion that most of the deformation occurs before the placement of wales and struts, and that it is unlikely that important arching in the horizontal direction will develop.

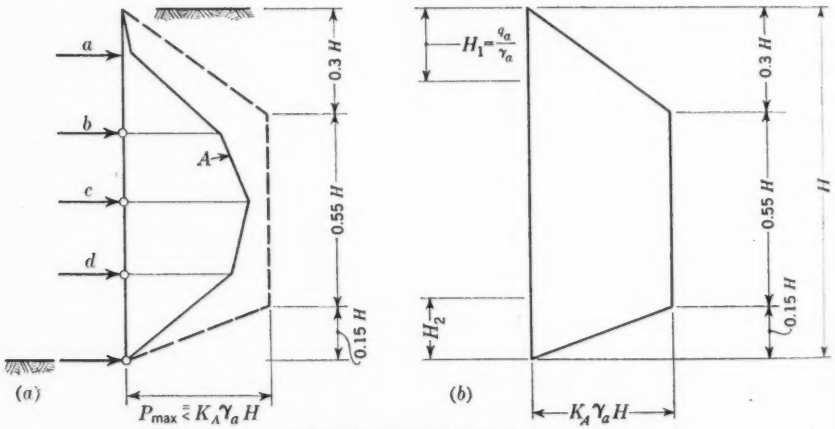
Inspection of Table 3(*b*) shows that the measured centers of pressure for the various open cuts were quite consistent, and were in general slightly below midheight of the cuts. In the cut S-4B the highest strut that was acted upon by earth pressure was 11 ft below the surface, and the next one at a depth of 19 ft. Hence the center of pressure while the cut was at shallow depths was necessarily low.

Table 1 gives the individual strut loads measured on contract S-1A. The deviation of the center of pressure from its average value is less than 15%. However, the variation in individual strut loads is very much greater. That this is due to minor details of construction was demonstrated in many instances such as the following:

[14] "A Fundamental Fallacy in Earth Pressure Computations," by K. Terzaghi, *Journal*, Boston Soc. of Civ. Engrs., April, 1936, p. 78.

On contract S-1A, excavation reached a depth of 24 ft in the bay between profiles 11 and 12 (see Table 1), and a strut at the 23-ft level was installed in each profile on the same day. In profile 11, excavation on the next day proceeded to a depth of 33 ft and a strut was placed immediately at the 31-ft level. In profile 12, excavation did not proceed from the 24 to the 33-ft level until the third day, at which time the entire bay to profile 13 was mined to 33 ft. The lower strut in profile 12 was not placed until 8 hr after the 33-ft depth was reached. As a result, the lower strut in profile 11 attained a final load of 232 kips, whereas the corresponding strut in profile 12 never exceeded 89 kips. Nevertheless, the center of pressure was at 0.43 H in both profiles.

It is apparent, therefore, that it is useless to attempt to compute the real distribution of lateral pressure over the sheeting. Of far greater practical importance is the statistical investigation of the variation in strut loads actually measured, in order to determine the maximum loads that may be expected under ordinary construction procedures. A useful method of investigation is by means of an equivalent beam, as demonstrated by Professor Terzaghi[15] in analyzing the results of measurements on the Berlin subway. The method is shown in Fig. 35. The vertical members of the sheeting are



Equivalent Beam for Strut Loads: (*a*) from Tests; and (*b*) for Design

FIG. 35

assumed to be hinged at each strut except the uppermost one, and a hinge is assumed to exist at the bottom of the cut. The abscissas of the pressure diagram "A" represent the intensity of horizontal pressure required to produce the measured strut loads. A study of such diagrams for all of the measured profiles disclosed that the maximum abscissa never exceeded the value $K_A \gamma_a H$. Every measured set of strut loads resulted in a different pressure diagram "A," all of which were found to lie within the boundaries of the trapezoid indicated by the dotted lines. Thus, if strut loads are computed on the basis

[15] "General Wedge Theory of Earth Pressure," by Karl Terzaghi, *Transactions*, Am. Soc. C. E., Vol. 106 (1941), p. 93.

of this trapezoid, they will most probably be on the safe side. The area of the trapezoid is approximately 50% in excess of the computed total lateral pressure, which is equal to

$$P_A = K_A \times \tfrac{1}{2}\,\gamma_a\,H^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (22)$$

per unit of length of the cut. The excess takes care of the statistical deviation of individual strut loads from the average.

Part V.—Settlement Considerations

The settlement of the ground surface is a direct result of the lateral squeeze of the clay toward the cut. Although construction conditions made it impossible to determine accurately the complete record of soil movements at the sides and bottom of the cut at the same profiles where complete settlement cross sections were available, a study of the partly incomplete results at numerous profiles established the approximate equivalence of the settlement volume and the volume of lateral squeeze. The most complete information at any of the measured profiles of movements of the sides and bottom of a cut is assembled in Figs. 25 and 26. In this particular cut, the presence of an existing freight tunnel, located as shown in Fig. 25, permitted measurements to be made of the lateral movement of the sheeting at the level of the tunnel while excavation had not yet reached the bottom of the cut. It is evident that the lateral squeeze extended for a considerable distance below the bottom of excavation.

The movements encountered at this profile are particularly interesting because they provide information from which can be deduced the maximum excavation that should be permitted before placement of the uppermost strut. On January 3, 1941, the earth pressure on the sheet piles was practically zero, because the trench, dug to permit driving the sheet piles, was not yet backfilled. In spite of this fact, the tops of the piling moved toward the cut more than 1 in. because of the unbalanced weight of the earth outside of the excavation. Most of this movement occurred while the depth of excavation increased from 10 to 13 ft, corresponding to an increase of the unbalanced part of the load from 1,100 to 1,440 lb per sq ft. The average unconfined compressive strength of the clay between a depth of 13 ft and the bottom of the cut was 1,340 lb per sq ft. Hence, the important deformation of the sheeting began when the unbalanced load became approximately equal to the unconfined compressive strength of the underlying clay. The disturbance of the clay was deep-seated, as evidenced by the appearance of horizontal cracks in the side walls of the freight tunnel when excavation reached a depth of approximately 8 ft. It seems possible that the abrupt and extensive yielding represents the spontaneous change in the clay from its relatively rigid undisturbed state to a plastic state. Installation of the vertical strut before reaching a depth H_1 equal to

$$H_1 = \frac{q_c}{\gamma_a} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (23)$$

would probably have resulted in the prevention of this sudden change and effected a reduction in the corresponding settlement. The struts actually

placed at a depth of 4 ft after excavation had reached 13 ft never received an appreciable load. On the other hand, in contract S-3, which was adjacent to contract S-1A with practically identical soil conditions, the top struts were placed when excavation reached a depth of only 8 ft. These struts experienced a substantial load, and no such sudden increase in lateral deflection of the sheeting, or of settlement, occurred. As a matter of fact, in every other open cut observed, the top strut was placed before a depth equal to H_1 was reached, and in each of these cases, the spontaneous settlement was absent. On contract D-6E, the soil was excavated 13 ft before struts were placed, exactly as in S-1A, but the value of H_1 for this case was 18 ft. The maximum movement experienced by the piling for the complete excavation of this cut was 0.5 in.

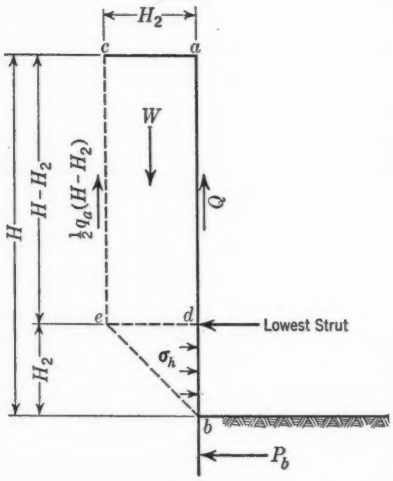


FIG. 36

The maximum permissible spacing between the bottom strut and the bottom of the excavation is less simple to formulate. No satisfactory analytical formulation has as yet been devised, which gives results in close accordance with the observations. The factors involved may be recognized by study of the simplified diagram in Fig. 36.

The total vertical force acting on a horizontal section (e d) at the level of the lowest strut is equal to the weight W of the prism a d e c minus the shearing resistance along c e, which is equal approximately to $\frac{1}{2} q_a (H - H_2)$, and minus the vertical load Q carried by the sheeting. The width of the prism is taken as H_2, equal to the distance between the lowest strut and the bottom of the cut, because the failure surface e b is likely to be inclined at 45° when $\phi = 0$. The intensity of vertical pressure at midheight of the space between b and d is, therefore,

$$\sigma_v = (H - H_2)\,\gamma_a - \frac{1}{2} q_a \frac{(H - H_2)}{H_2} - \frac{Q}{H_2} + \frac{1}{2}\gamma_c H_2 \ldots\ldots (24)$$

in which: γ_a = average density of material above lowest strut; γ_c = average density of material between b and d; and q_a = average effective unconfined compressive strength of material above lowest strut. The lowest value of the normal stress σ_h on the vertical face b d which is compatible with the average vertical stress σ_v is

$$\sigma_h = \sigma_v - q_c = (H - H_2)\,\gamma_a - \frac{1}{2} q_a \frac{(H - H_2)}{H_2} - \frac{Q}{H_2} + \frac{1}{2}\gamma_c H_2 - q_c \ldots (25)$$

in which q_c = unconfined compressive strength of material between b and d.

For the particular case when the entire vertical load is carried by shearing stresses on the wall and on the plane c e, the value of σ_h becomes equal to

zero (the most desirable condition) when

$$H_2 = \frac{2\,q_c}{\gamma_c} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

or, in other words, when the distance below the bottom strut equals the critical height of the material. (The critical height is the theoretical height at which a vertical bank of cohesive material will stand without support.)[16] If the vertical load is greater than zero, or if it is desired to increase H_2 beyond the critical height of the material, it is necessary that the vertical sheeting be stiff enough to carry the horizontal load σ_h, and that the clay beneath the bottom of the cut be adequate to carry the reaction P_b with sufficient factor of safety. Calculation for the Chicago open cuts indicates that the method of Eq. 23 is likely to be ultraconservative, but it is seen that until measurements are available for evaluation of the shearing forces on the sheeting, a theoretical solution, even of more refined nature, would not be of great practical value.

Repeated calculations of the distances H_2 as excavation and successive struts advanced would determine the maximum vertical spacing between struts in order to prevent excessive lateral squeeze and settlement. This distance was empirically checked for the open cuts on the Chicago subway by systematic observations of inward movement of the sheeting. Excessive movements were readily determined and corrected by relocating struts. An instructive example was again furnished by contract S-1A. Between a depth of 24 and 34 ft, the critical height of the clay ranged between 10 and 15 ft. In the first part of the cut, the vertical distance between the second and third row of struts, at depths of 13 and 31 ft, was equal to 18 ft, considerably greater than the critical height. The corresponding lateral movement and settlement were very important. As a consequence, an intermediate row of struts was established which reduced the spacing at the softest part of the clay to 8 ft. Thereafter the lateral squeeze dropped from $3\frac{1}{4}$ to $2\frac{1}{4}$ in. at this elevation, and the settlement on adjacent buildings was reduced approximately 35%. The total pressure on all the struts of one vertical row remained practically identical.

On one early open cut, 46 ft deep, upon which measurements were not made, a lateral yield of 10 in. developed, and evidence of considerable distress in some of the bracing appeared. From this observation, together with those already described, it may be concluded that for the Chicago clays, a lateral yield of slightly less than 0.25% of the height of the cut is all that is necessary to reduce the earth pressure to the active value, and that the magnitude of the pressure remains substantially unchanged for a yield up to 1%. Beyond this value, the lateral pressure again increases. Furthermore, it is apparent that nature takes care that a yield of at least 0.25% will occur, regardless of the type of bracing.

Part VI.—Rules for Design

For open cuts constructed in a normally consolidated clay soil in accordance with standard construction procedure, the following rules for design seem justified on the basis of the Chicago subway experiments:

[16] See "Poussée des Terres," Pt. II, by J. Résal, Paris, 1910, p. 77.

1. From test borings, secure samples for unconfined compression tests and other soil data needed to establish the value q in the equation (compare Eq. 18*a*)

$$q_a = \frac{1}{H}\left[\gamma_1 H^2_s K_s \tan \phi_1 + (H - H_s)\, n\, q_c\right] \ldots\ldots\ldots (27a)$$

and for clays with no overlying sand layers,

$$q = n\, q_c \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (27b)$$

For Chicago, $n = 0.75$. For other locations, the value of n must be determined on the basis of a few representative field tests.

2. Compute the total lateral pressure by use of the formula

$$K_A = 1.2\left(1 - \frac{2\, q_a}{\gamma_a H}\right) \ldots\ldots\ldots\ldots\ldots\ldots (28)$$

The factor 1.2 in Eq. 28 merely takes account of the scattering from the mean value. It should not be taken to indicate that K_A can be greater then 1.0 for a perfect fluid.

3. Construct the pressure diagram Fig. 35, with maximum abscissa, $K_A\, \gamma_a\, H$, and compute the maximum probable strut loads. The struts may be designed for these loads with a conservative factor of safety, inasmuch as provision for all normal variations has been made in the equations. Wales should be designed with no reduction for horizontal arching.

4. Determine allowable depth of excavation prior to placing the first row of struts, from Eq. 23—$H_1 = \frac{q_c}{\gamma_a}$.

Conclusive information concerning the maximum permissible vertical spacing between remaining struts has not been obtained. The proper depth of penetration for the piling depends to a large extent on the stiffness of the clay, and the change in stiffness below the bottom of the cut. Therefore, no general rules can be established for determining this depth.

It should be cautioned that although the results of the Chicago tests are likely to be applicable to other deposits of normally consolidated clays, they almost surely will not apply to clays which have been overconsolidated by heavy pressures in their previous geological history, nor to clays possessing cracks and fissures. They are not likely to apply to shafts or to other excavations in which the deformation conditions of the sheeting are likely to be restricted or significantly different from those in the excavations described. Although the general applicability of Eqs. 11 for total pressure in the Chicago clays has been strongly indicated, the final test in any other locality resides in field measurements.

EARTH PRESSURE ON TUNNELS

By W. S. Housel,[17] M. Am. Soc. C. E.

Synopsis

An investigation of earth pressure on tunnels was undertaken in Detroit, Mich., in 1930, when a number of pressure cells were installed in tunnels, shafts, and retaining walls. Goldbeck pressure cells were used, and practically all installations were in structures built in or on the plastic clay deposits typical of this and several other cities bordering on the Great Lakes.

Earth pressures were observed for periods varying from a few months to several years. Some installations became inaccessible, as the structures were placed in service, and some others were destroyed. In other cases it has been possible to extend the measurements over longer periods, and for the Central Avenue Tunnel, in Detroit, which is discussed in this paper, the records have been continued to the present time, covering a period of approximately ten years.

The compilation and analysis of this particular set of observations, supplemented by shearing resistance tests at the location of the tunnel, were completed in September, 1941. During the years that the data were accumulating, and particularly during the two years 1940 and 1941, a number of analyses have been made in an attempt to develop a logical and accurate method of representing the pressure variations that were observed. In this way, gradually, the "circuit method" of computing earth pressure on tunnels has been evolved. This method consists of establishing the static equilibrium of adjacent elements of mass at the top, side, and bottom of the tunnel in terms of the shearing resistance of the soil. The circuit is then closed by checking from the top to the bottom of the tunnel, treating the tunnel itself as a free body in equilibrium under the combination of stresses conceived to be acting in the surrounding mass.

When such a circuit is closed it means simply that the equations of static equilibrium have been completely satisfied by the selected system of stresses. To be of practical value such a method must be capable of reproducing the pressure variations that have been observed and must do so in terms of measurable soil properties. It is the purpose of this paper to present the experimental background of such a method, develop it, and demonstrate its applicability under the observed conditions.

The extent to which the method may be applied to the computation of

[17] Research Consultant, State Highway Dept.; Associate Prof., Civ. Eng., Univ. of Michigan, Ann Arbor, Mich.

earth pressure under more general conditions must be demonstrated by other applications. In so far as the fundamental basis is nothing more than the application of the laws of static equilibrium, general applicability scarcely can be denied. The concept used to represent soil resistance, however, does involve a viewpoint that has been the subject of considerable deliberation and debate. In this connection it should be stated that the paper is confined to a discussion of plastic clay. According to the generally accepted definition of a plastic solid, the intent, therefore, is to discuss materials in which shearing resistance is a function of true cohesion and independent of internal friction. In other words, it is assumed that the soil involved in this investigation is a purely cohesive material. Discussion of the method and its general applicability doubtless will be focused largely on the question of whether or not soil resistance may be so represented, and, if so, how wide a range of soil texture may be included in that classification.

Experimental Results

A pressure-cell installation at the Central Avenue Tunnel has produced a consistent series of earth-pressure measurements extending over a long enough period of time to give a complete picture of the changing condition of equilibrium in such a disturbed soil mass. Soil conditions were quite uniform, there were no apparent disturbances other than the tunnel construction, and the use of the tunnel was such as to permit continued observation for an unlimited time. Other installations were complicated by nonuniform soil conditions, other major excavations in the immediate vicinity, or restrictions on the period of observation.

Fig. 37 shows the general construction features of the tunnel, which is of monolithic concrete with an inside diameter of 9.5 ft and an outside diameter slightly less than 13 ft. The tunnel was built as a by-pass for a large water main beneath a main-line railroad. The soil conditions as indicated by four original borings were quite uniform and consisted of approximately 12 ft of sand and fill at the surface, underlain by a plastic blue lacustrine clay for the entire depth to hardpan. A fifth boring was made in 1940 to obtain samples for shearing resistance tests, the results of which will be used in the present study. The tunnel was located at considerable depth, there being approximately 60 ft of cover. Three rings of pressure cells were installed at the location shown in Fig. 37, all within a distance of some 200 ft.

The arrangement of the cells in the three sections is shown in Fig. 38. The various borings indicated that the soil conditions were substantially uniform, so that it is possible to combine the results to give a desirable duplication of measured pressures. The composite diagram of pressure-cell locations (Fig. 38(*d*)) indicates that the total of twenty-two cells provides three duplicate readings at the top and bottom of the tunnel, five at the spring line, and, with one exception, two readings at each of six intermediate points.

Shearing Resistance Tests.—In Fig. 39 are shown the results of the shearing resistance tests plotted as a shear profile. After penetration through the sand overburden, samples were taken every 5 ft down to hardpan at a depth considerably below the tunnel. The shearing resistance as measured by these

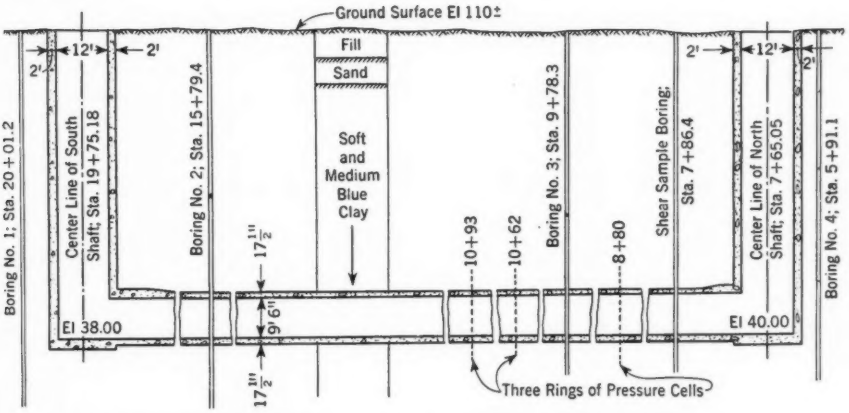


FIG. 37.—GENERAL SECTION OF CENTRAL AVENUE TUNNEL, DETROIT, MICH.

tests was quite uniform although there was some variation due, presumably, to the stratification characteristics of clay deposits in this area. The shear varied from approximately 400 lb per sq ft at the top of the clay to an average of 150 lb per sq ft for the remainder of the depth to the top of the tunnel. From the top of the tunnel to hardpan the clay was somewhat stiffer, the average being taken as 200 lb per sq ft. The shearing resistance of the sand overburden has been estimated as equivalent to the weight transfer for a 30° sand arch or wedge.

The shearing resistance of the clay was measured by the ring-shear test which has been described in detail in other publications.[18,19] This type of test uses static loading, and the shearing resistance is measured without the application of any pressure normal to the shear plane. The test thus conforms strictly to the accepted definition of a purely cohesive material without internal friction. In this connection the volumetric analyses of the samples are shown in Fig. 39. It may be noted that the voids varied from 40% to 55% by volume, and these voids were filled with water as the air voids were practically negligible.

The heavy dashed line in Fig. 39 represents an average shear profile designated as the design value shearing resistance from which are taken shear values used in the subsequent analysis of earth pressure. These values conform to the yield value as ordinarily defined and determined by the adopted test procedure. There are two significant figures taken from this shear profile which will be used subsequently. These are (1) the shearing resistance at the level of the tunnel, which is taken as 200 lb per sq ft, and (2) the average vertical

[18] "The Shearing Resistance of Soil—Its Measurement and Practical Significance," by W. S. Housel, *Proceedings*, A.S.T.M., Vol. 39, 1939, p. 1084.

[19] "Laboratory Manual of Soil Testing Procedures," published by Edwards Brothers, Ann Arbor, Mich., 1939.

shear from the surface of the ground to the top of the tunnel, which is approximately 300 lb per sq ft.

Earth-Pressure Measurements.—The results of the earth-pressure measurements are shown in Fig. 40, the pressure-cell readings being plotted in polar

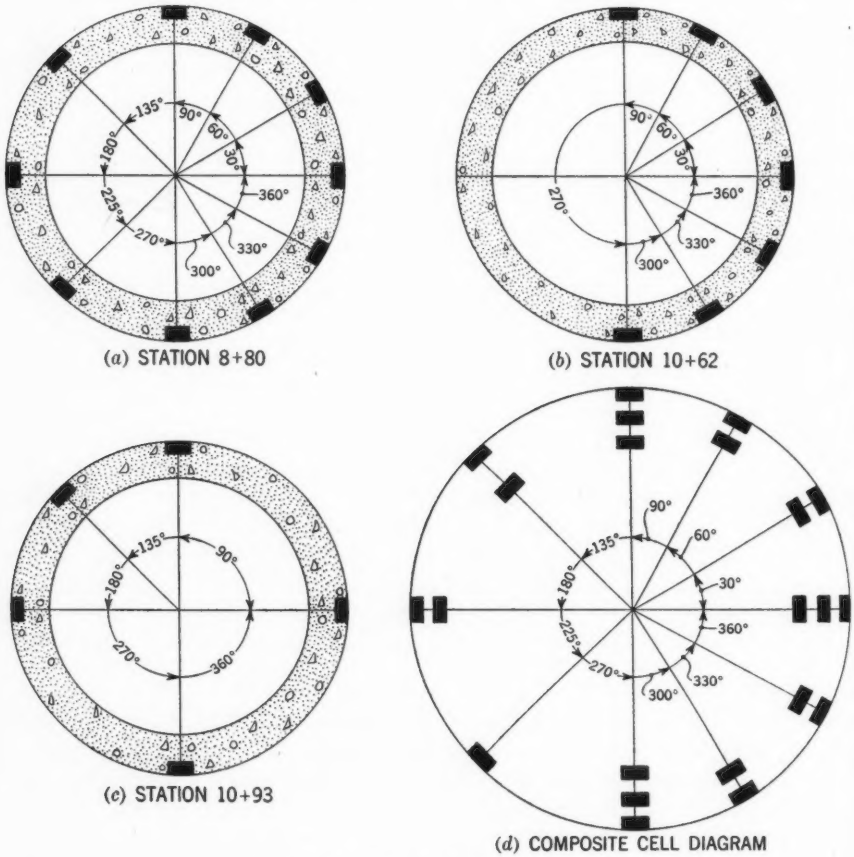


FIG. 38.—LOCATION OF PRESSURE CELLS

coordinates. The pressure diagrams or pressure rings are plotted to a scale of 1 in. = 4,000 lb per sq ft so that the two circles shown represent 4,000 and 8,000 lb per sq ft, the center being the origin. The individual pressure readings are represented by indicating the range of the readings at each point on the circumference, with the average values also designated. The numbers shown in small circles indicate the number of readings at each point that are included in the average.

At the first two dates (December 4 and December 10, 1930) an air pressure of 27 lb per sq in. was being carried in the tunnel. The pressure readings and particularly the increase between those two dates would indicate that air

pressure was not sufficient to balance the active earth pressure completely and that gradual yielding was taking place. It should be noted that the pressure readings were made inside the tunnel with cells and measuring devices referred to the existing air pressure as a datum. The pressures recorded by the cells

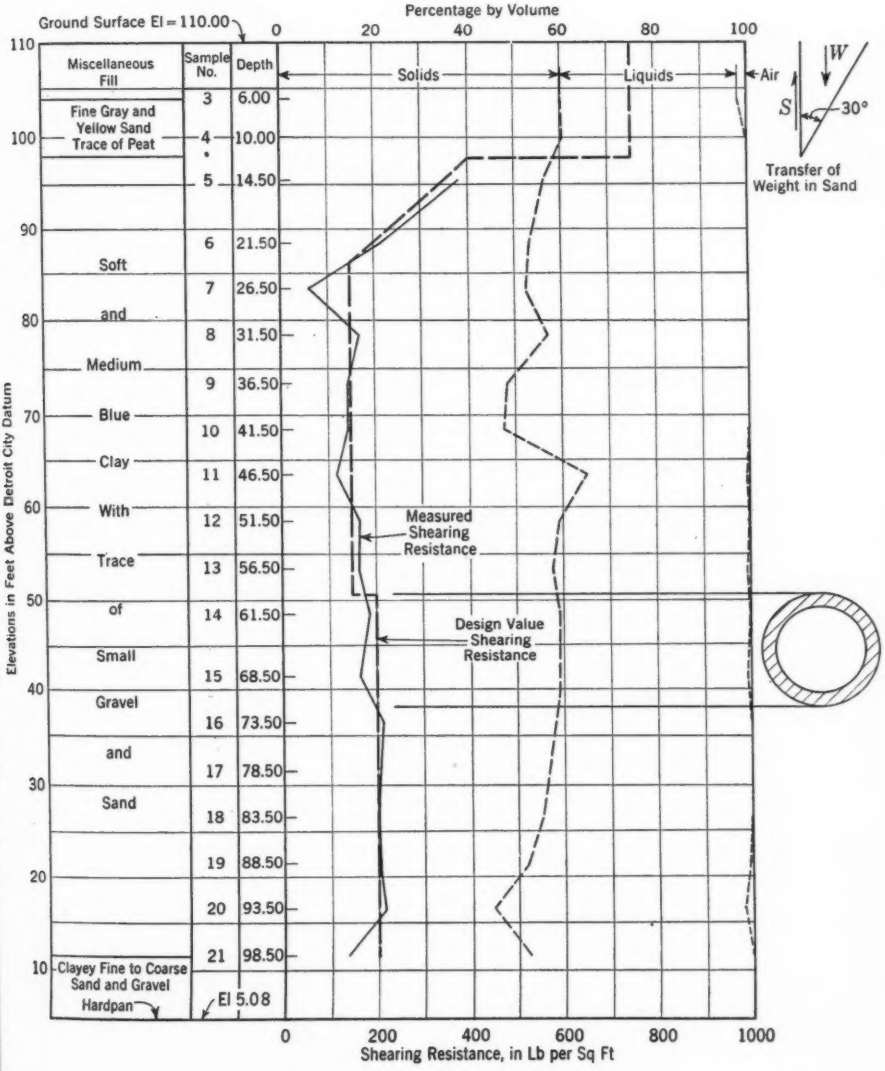
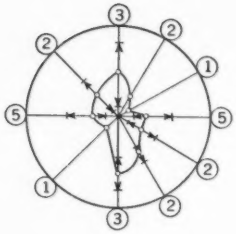


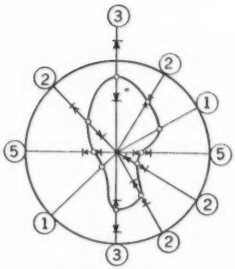
FIG. 39.—SHEARING RESISTANCE TESTS

previous to December 12, 1930, are earth pressures in excess of the pressure maintained in the tunnel.

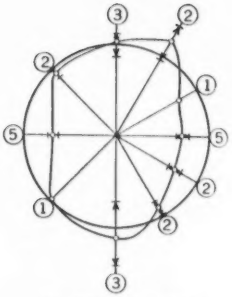
Starting on December 10, 1930, and for the succeeding two days, the com-



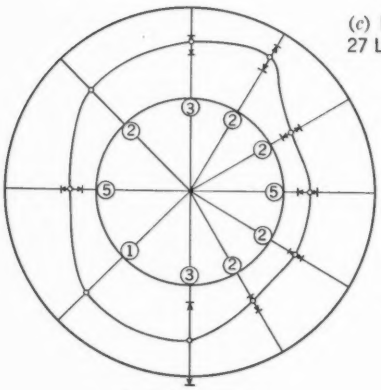
(*a*) DEC. 4, 1930
27 Lb Air in Tunnel



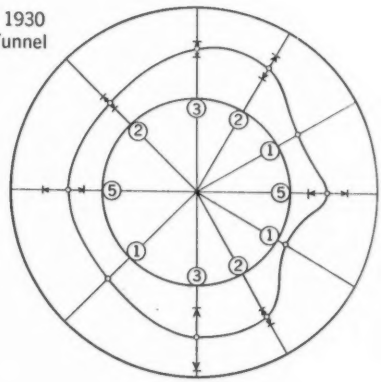
(*c*) DEC. 10, 1930
27 Lb Air in Tunnel



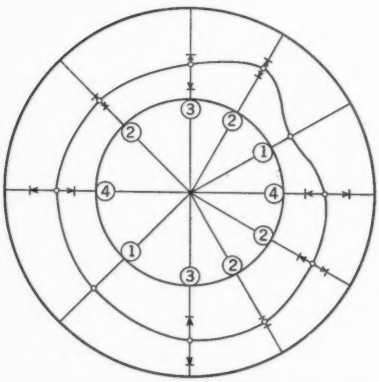
(*b*) DEC. 11, 1930
16 Lb Air in Tunnel



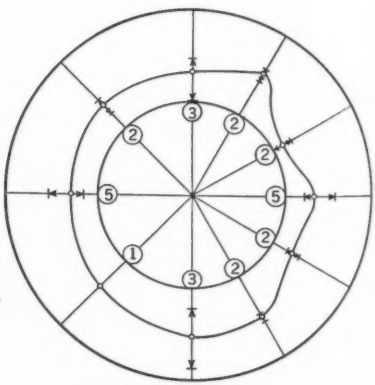
(*d*) DEC. 12, 1930
No Air Pressure in Tunnel
on or After This Date



(*e*) DEC. 31, 1930

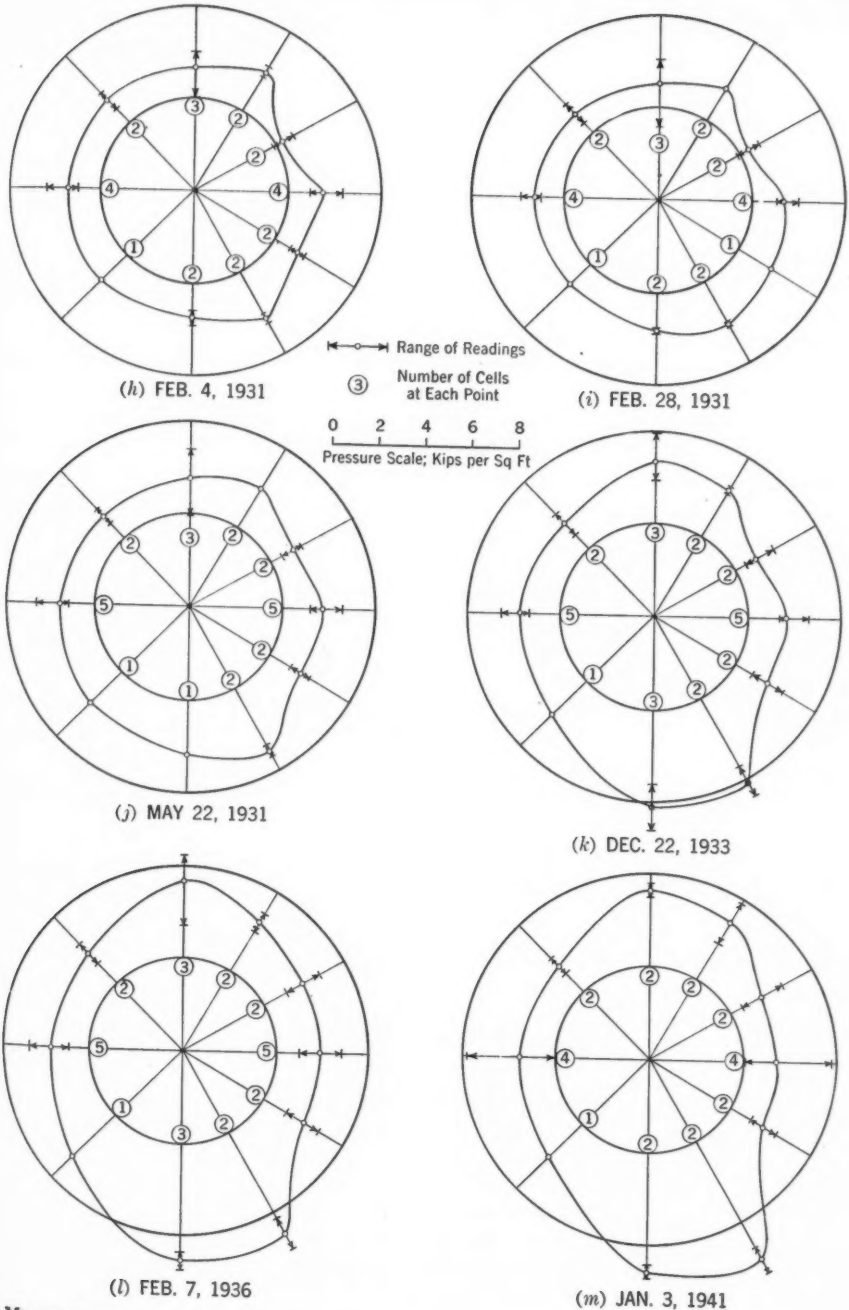


(*f*) JAN. 13, 1931



(*g*) JAN. 16, 1931

FIG. 40.—EARTH-PRESSURE



(*h*) FEB. 4, 1931

(*i*) FEB. 28, 1931

(*j*) MAY 22, 1931

(*k*) DEC. 22, 1933

(*l*) FEB. 7, 1936

(*m*) JAN. 3, 1941

pressed air was released, and air pressure in the tunnel returned to zero or atmospheric pressure. The immediate change in earth pressure recorded by the cells is significant and was approximately equal to the change in air pressure. After the release of the compressed air a period of slow change in pressures occurred over comparatively long periods of time. Figs. 40(*f*) to 40(*j*), covering a period of six months after construction, show relatively small changes. Particular attention is directed to the date of February 28, 1931, at which time the pressures around the tunnel circumference were approximately equal. This date represents the end of a first stage of variation and is thus a control point in the subsequent analysis. After that date there is a gradual increase in top and bottom pressures, culminating in the final pressure diagrams shown in Figs. 40(*k*), 40(*l*), and 40(*m*).

A significant change in pressure was recorded until 1936, approximately five years after construction. In 1941, approximately ten years after construction, most of the cells were still in operation, although outlet pipes and electrical connections were in rather poor condition due to moisture and corrosion. Readings were a little more erratic as shown by the wider range for individual cells. The same pressure pattern was present, however, as that found in 1936 and, except for a slight decrease in lateral pressure at the spring line and an increase in the pressure at one point on the invert, there were no significant changes in the last five years of observation.

The variation in pressure with time is shown much more clearly in Fig. 41, in which measured pressures at the top, side, and bottom of the tunnel are shown for the full 10-yr period. When the compressed air was released, the immediate change in the top vertical pressure was 3,200, side pressure 4,100, and bottom vertical pressure 4,000 lb per sq ft. The fact that the average change was approximately equal to the drop in air pressure of 3,880 lb per sq ft indicates the relative accuracy of Goldbeck pressure cells and is evidence that the experiment was yielding consistent results.

In Fig. 41 the static head at the top, side, and bottom of the tunnel has been shown by horizontal lines to serve as a background for the comparative analysis of measured pressures. The two stages of pressure variation previously mentioned are now clearly apparent. For a period of approximately three months after the release of the air pressure, up to February 28, 1931, all pressure readings were decreasing and, although not quite equal at all points, the difference from top to bottom of the tunnel was comparatively small. After this first stage of decreasing pressures, there was a period of several years during which the top and bottom vertical pressures were constantly increasing up to values approximately equal to the static head at the respective elevations. According to the observations, there were substantial changes in these pressures for as long as five years after construction. At that time a final and permanent condition of equilibrium appears to have been achieved, as there were no significant changes for the next five years up to 1941.

During the progressive increase in vertical pressure up to static head, the lateral pressure at the side of the tunnel remained approximately constant and very much less than the static head at that elevation. Thus, in Fig. 41 is shown a consistent variation in earth pressure that is indicative of changing

conditions of equilibrium in the earth mass for a period of at least several years after construction. There appear to be two distinct stages of behavior. The first involves a temporary condition of equilibrium during which it may be presumed that maximum resistances in the mass are being mobilized to transfer weight away from the tunnel. The second stage of increasing vertical pres-

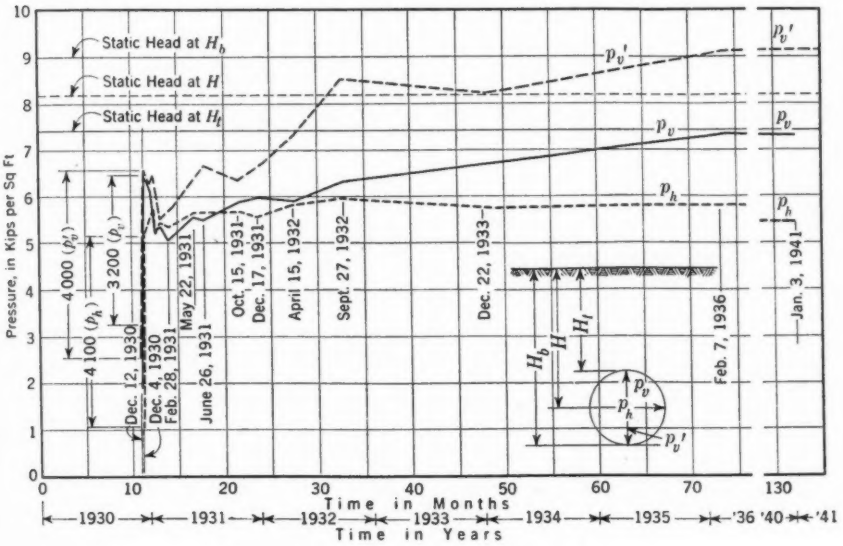


FIG. 41.—VARIATION IN PRESSURE WITH TIME

sures, culminating in final equilibrium, represents some transfer of weight back to the tunnel.

In the first stage, the changes observed show that the weight transfer under temporary equilibrium was greater than could be maintained permanently. This phenomenon presents evidence that dynamic soil resistance has been temporarily brought into play and is an important factor to be considered in an analysis of results. Throughout the second stage of behavior, the side of the tunnel is shielded against the full static earth pressure and remains the point of minimum active pressure. The fact that this lateral pressure is substantially less than static head after final equilibrium indicates that there are permanent or residual shearing stresses in the mass as a result of the disturbance of its internal equilibrium. The fact that the tunnel structure weighs less than the displaced earth by approximately 5,300 lb per lin ft also indicates that there must be some permanent changes in the conditions of equilibrium in the soil mass.

An analysis of the earth-pressure measurements must explain the manner in which weight is transferred away from the tunnel during and for some time after construction and then is shifted back before coming to final equilibrium. Fig. 42 is an attempt to picture this weight transfer schematically in terms of certain conditions that must be involved. In Fig. 42(*a*) conditions are indi-

cated that may be considered as controlling during the earlier stage of temporary equilibrium. It is conceived that displacement is toward the tunnel from all directions, with soil resistance being mobilized on all sides in opposition to this motion. Thus, weight is being transferred from the entire region above and on either side of the tunnel. Under these circumstances active pressure

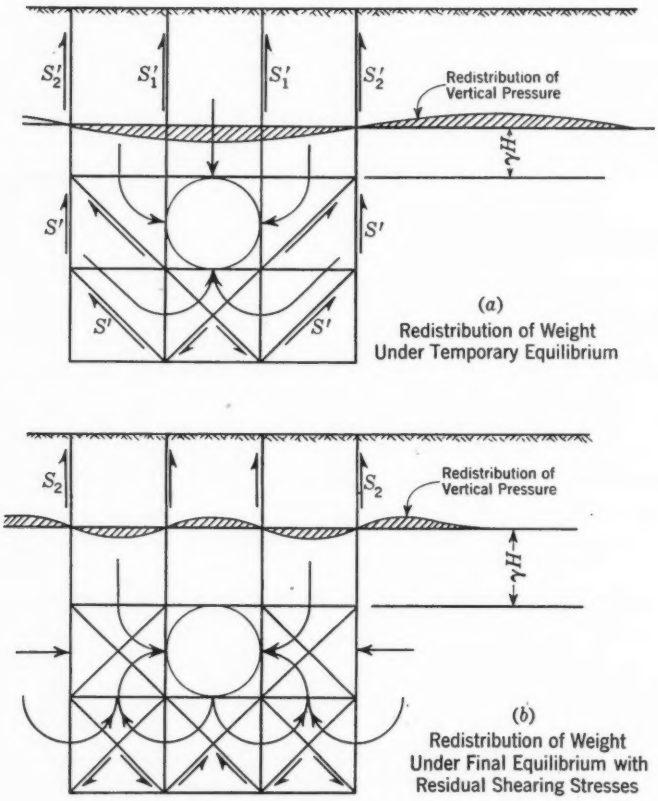


FIG. 42.—WEIGHT TRANSFER IN TUNNEL EXCAVATION

tends toward minimum values at all points on the tunnel circumference as the mobilized shearing resistance approaches maximum. A considerable part of this resistance is dynamic and in accordance with the properties of plastic solids in which the shearing stress in excess of the yield value is proportional to the rate of movement. This trend, as previously shown, reaches a limit some three months after construction, at which time controlling values may be established.

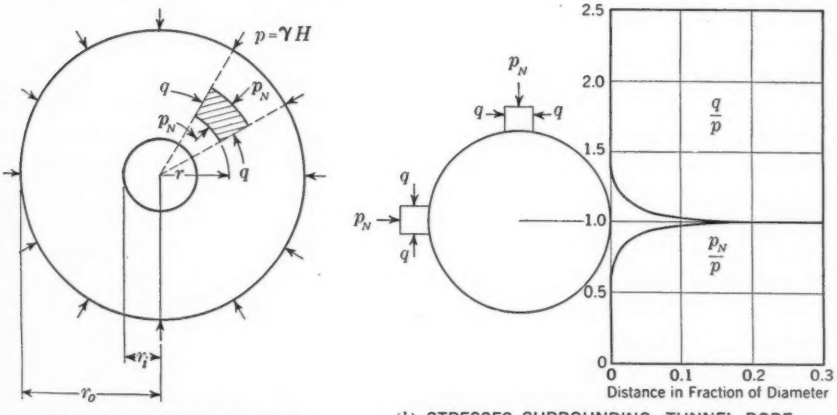
Fig. 42(*b*) shows conditions conceived to exist as a final condition of equilibrium. In the first place, the shearing resistance values have decreased to the yield value which is the maximum that may be maintained under static equilibrium. Thus, due to subsidence on either side of the tunnel, some weight

has been transferred back to the tunnel, building up vertical pressure at the top and bottom. The lateral pressure at the side is still minimum and, as a matter of fact, has been almost constant at all times. In setting up a system of stresses for the final condition of equilibrium, it is assumed that any tendency for motion would be toward the sides of the tunnel from points of high pressure at the top and bottom, as well as from static pressure outside of the elements adjacent to the tunnel.

Analysis of Earth-Pressure Measurements

The basis for analyzing the earth-pressure measurements has been portrayed by the preceding discussion of experimental data. Aside from the data themselves, significant trends or stages of variation have been indicated. These trends have been interpreted in an attempt to describe the changing conditions of internal equilibrium in the soil mass. There remains now the task of setting up specific equations of static equilibrium from which the magnitude of shearing stresses may be estimated and with which actual shearing resistance may be compared.

As a first approach to the stresses in the soil mass surrounding the tunnel, it is enlightening to review a related problem from elastic theory. Fig. 43 shows the relationships involved in computing the stresses in a thick cylinder subjected to a uniform external pressure. Using the notation shown in Fig. 43,



(*a*) STRESSES IN THICK CYLINDER (*b*) STRESSES SURROUNDING TUNNEL BORE ELASTIC THEORY

Fig. 43.—Stresses in a Thick Cylinder

S. Timoshenko[20] gives the maximum and minimum normal stresses, p_N and q_m, respectively, as:

$$p_N = \frac{p\,r^2_o}{r^2_o - r^2_i}\left(\frac{r^2_i}{r^2} - 1\right) \ldots\ldots\ldots\ldots\ldots\ldots (29a)$$

and

$$q_m = \frac{p\,r^2_o}{r^2_o - r^2_i}\left(1 + \frac{r^2_i}{r^2}\right) \ldots\ldots\ldots\ldots\ldots\ldots (29b)$$

[20] "Strength of Materials," by S. Timoshenko, Pt. II, D. Van Nostrand Co., 1930.

When r_o is equal to infinity and r is equal to r_i, as would be the case at the surface of a tunnel bore in an indefinite mass, these equations reduce to $p_N = 0$ and $q_m = 2\,p$.

The maximum shearing stress is equal to one half the difference in the principal stresses and is thus equal to static pressure, p. Applying these equations to the tunnel problem also reveals (as shown in Fig. 43(*b*)) that the principal stresses become practically equal and the shearing stress becomes zero, within a distance from the surface of the tunnel bore of less than one fourth of the diameter of the tunnel.

Several important deductions may be made from this elastic solution. In the first place, the pressure acting normal to the tunnel bore is a principal pressure, from which it follows that under normal conditions there would be no tangential or shearing forces on this surface. In the second place, at the surface of the tunnel bore there is the usual principal stress difference, equal to twice the shearing resistance of the material. When movement of the soil mass is toward the tunnel, the normal pressure would be the minimum principal stress. In the third place, the stress differential at the surface of the tunnel is dissipated within a relatively short distance from the tunnel.

The stress relationships from elastic theory obviously hold only as long as the shearing resistance of the material involved is not exceeded. In plastic clay, shearing resistance is frequently of a low order of magnitude as compared to the static earth pressures that are encountered in ordinary construction practice. For example, in the present case the static pressures vary from 7,000 to 9,000 lb per sq ft, whereas the shearing resistance is from 200 to 300 lb per sq ft.

From the elastic solution it was determined that the maximum shearing stress in the case of an unsupported tunnel would be equal to the static pressure. Actually, yielding of the soil would take place in the soil mass almost immediately at the surface of the tunnel bore, and would extend back through the mass until sufficient resistance was mobilized to furnish a reaction to absorb the unbalanced pressure. Because the shearing resistance is relatively small, the unbalanced pressure generally will require artificial support. The reaction furnished by the soil mass will be developed by displacements in the range of plastic, rather than elastic, deformation.

Because of plastic flow or "squeezing," shear is mobilized on shear planes or surfaces throughout the mass, resulting in the weight transfer which reduces the active pressures substantially below the static head. The total reaction developed depends upon the shearing resistance of the soil, the diameter of the tunnel, the depth beneath the surface, and the weight of the soil. With these various factors determined, the analysis of earth pressure can be reduced to the selection of a stress system consistent with static equilibrium, which is capable of reproducing the measured pressures.

Such a system of stresses is shown in Fig. 44 and constitutes what has been previously called the "circuit method" of computing earth pressure on the tunnel lining. For the temporary condition of equilibrium the stress system shown involves the maximum transfer of weight away from the tunnel.

For the top vertical pressure (Fig. 44(*a*)):

$$p_v = \gamma H_t - \frac{2 S_1 H_t}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

in which $S_1 = C_1 S_v$; S_v is the cumulative average shear from the ground surface to the top of the tunnel; and γ is the soil weight per cubic foot.

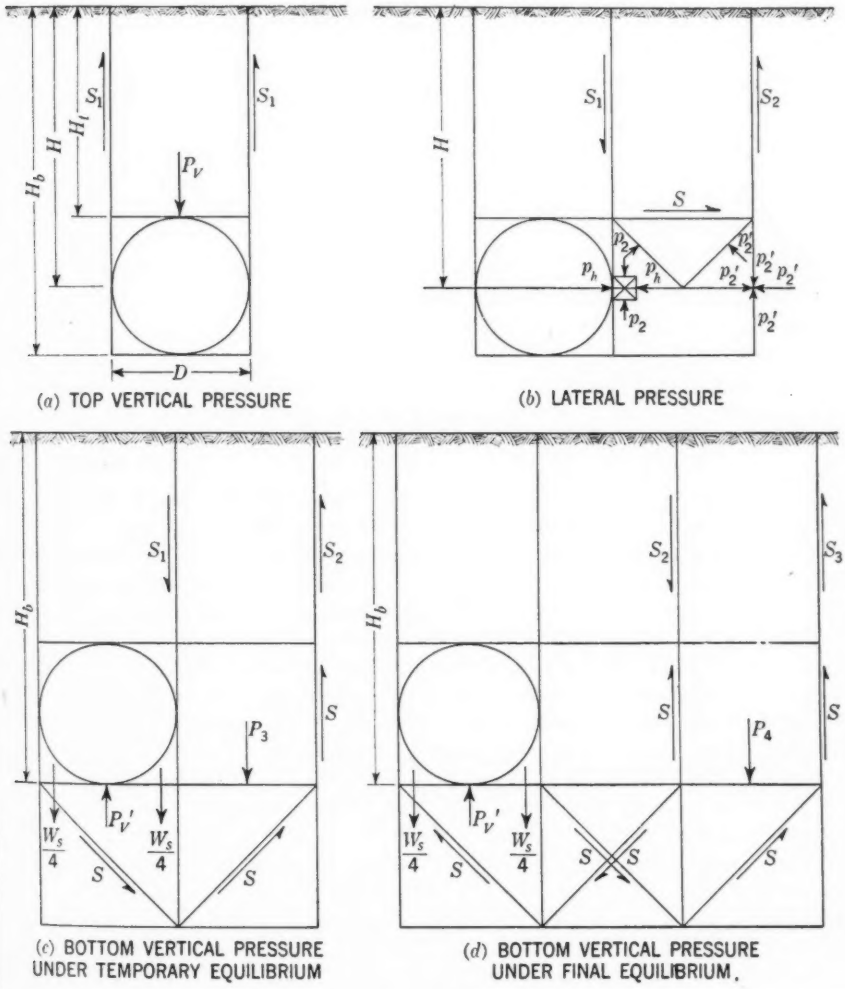


FIG. 44.—COMPUTATION OF EARTH PRESSURE

For lateral pressure (Fig. 44(*b*)):

$$p'_2 = \gamma H + \frac{S_1 H_t}{D} - \frac{S_2 H_t}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (31)$$

in which $S_2 = C_2 S_v$; $S = C_2 S_A$; and S_A = average shear in the region of the tunnel. If $p_2 = p'_2 - 2S$ and $p_h = p_2 - 2S$, Eq. 31 becomes

$$p_h = \gamma H + \frac{S_1 H_t}{D} - \frac{S_2 H_t}{D} - 4S \quad \ldots\ldots (32)$$

The bottom vertical pressure under temporary equilibrium (Fig. 44(*c*)) is found by

$$p_3 = \gamma H_b + \frac{S_1 H_t}{D} - \frac{S_2 H_t}{D} - S \quad \ldots\ldots (33)$$

and

$$p'_v = p_3 - 4S - \frac{W_s}{2} = \gamma H_b + \frac{S_1 H_t}{D} - \frac{S_2 H_t}{D} - 5S - \frac{W_s}{2} \quad \ldots (34a)$$

As a check:

$$p'_v = p_v + W_t + \frac{W_s}{2} \quad \ldots\ldots (34b)$$

In Eqs. 34, W_t is the total weight of the tunnel and $\frac{W_s}{4}$ is the total weight of the earth in a corner fillet (Fig. 44(*a*)).

Although Fig. 44 and Eqs. 30 to 34 are more or less self-explanatory, there are certain features that require some elaboration. The size of the elements of mass for which the equations are written has been taken equal to the diameter of the tunnel. Thus, elements surrounding the tunnel are taken as cubes of finite rather than infinitesimal dimensions. It is in this way that dimensional factors are brought into the solution and full recognition accorded the discontinuities in stress which arise from boundary effects and inability of the soil to withstand the potential stresses that are involved.

Although this method permits important simplification, it also introduces approximations which must be accepted. The equations are set up as if the tunnel bore was square, rather than circular, and thus some of the effects of the corner fillets are neglected. Stresses that are pictured as acting on elements of the same size as the tunnel bore are the average of actual stresses which vary over the finite dimensions of the element. Thus, in devising a method of dealing with stress discontinuities that cannot be incorporated in conventional stress relationships, it becomes impossible to work in terms of continuous stress variations, which are the more familiar. The validity of such approximations is difficult to prove or disprove except by the final test of whether or not they reproduce the measured pressures.

The equations of equilibrium involve the combination of three general factors of resistance. First, there is the weight transfer due to shearing resistance mobilized on vertical planes extending from the top of the tunnel to the surface of the ground. This transmission of force around the tunnel bore is sometimes referred to simply as arching action. The second influence to be considered likewise may be referred to as arching of the lateral pressure around the tunnel. It is shown, however, as shear on a horizontal plane at the top of the tunnel.

Both of these factors are functions of plastic displacements and the resulting mobilization of shearing resistance. They both operate to diminish the

static earth pressure in the region of the tunnel, but they have been evaluated in two separate steps. The static pressure at the depth H is shown at the back of the vertical column that is affected by lateral movement toward the tunnel. It is computed by evaluating the magnitude of the weight transfer by shearing forces on vertical planes of slipping. The horizontal arching action is then evaluated as a pressure differential made available by mobilization of horizontal shearing resistance. Finally, there is the third factor of resistance which makes available a further decrease in lateral pressure. This is the principal stress differential at the surface of the tunnel bore as previously deduced from the elastic solution based on stresses in a thick cylinder. Final equations are then written for the summation of all resistance factors. As stated, Eq. 30 is for the vertical pressure at the top, and Eq. 32 is for the lateral pressure at the side.

Eq. 34*a* for the bottom vertical pressure is derived by extending the same method of analysis to an element of mass below the tunnel. Resistance to upheaval at the bottom of the tunnel includes the summation of shearing resistance mobilized on the sides of the vertical column beside the tunnel, which tends to move down when upheaval at the bottom of the tunnel is impending. In addition, there is the pressure differential on two elements below the tunnel and the weight of the corner fillets. Eq. 34*b* is the check equation and closes the pressure circuit from top to bottom of the tunnel.

For final equilibrium, the equations for the top vertical and the lateral pressure on the side are the same as for temporary equilibrium (Eqs. 30 to 32). As previously stated, the final pressures represent the limiting case of a second stage of behavior in which weight is being transferred back to the tunnel. The top and bottom are now points of maximum pressure, and it is conceived that the tendency for motion is toward the side. As shown in Fig. 44(*d*), this tendency controls the bottom vertical pressure as expressed by

$$p_4 = \gamma H_b + \frac{S_2 H_t}{D} - \frac{S_3 H_t}{D} - 2S \quad \ldots\ldots\ldots\ldots\ldots\ldots (35)$$

Then

$$p'_v = p_4 - \frac{W_s}{2} = \gamma H_b + \frac{S_2 H_t}{D} - \frac{S_3 H_t}{D} - 2S - \frac{W_s}{2} \quad \ldots\ldots\ldots (36)$$

in which $S_3 = C_2 S_v$. As before, a check is afforded by application of Eq. 34*b*, which then closes this circuit from top to bottom of the tunnel. Eq. 36 differs substantially from Eq. 34*a* for the pressure at the same point in temporary equilibrium.

Relation of Shear Stress to Yield Value.—In all of the equilibrium equations the shearing stresses are set up in general terms, and specific values are yet to be established. The correlation of specific values of shearing stress and the shearing resistance of the soil as measured by test is the primary objective of the investigation. This correlation may be made most conveniently in terms of the ratio of shearing stresses computed from the equilibrium equations to the yield value of the soil as measured. This ratio has been designated by C_1 and C_2 and has been called the overload ratio, inasmuch as shearing stresses mobilized under field conditions are frequently in excess of the yield value.

The two controlling values of shearing resistance are 200 lb per sq ft for the average shear, S_A, at the depth of the tunnel, and 300 lb per sq ft for the average vertical shear, S_v, for the depth from the surface of the ground to the top of the tunnel. By substituting the measured values of earth pressure and shearing resistance in the equilibrium equations, the values of C_1 and C_2 may be computed for the entire range of observation. It should be noted that C_1 refers to the shearing stress at the sides of a vertical column above the tunnel, whereas C_2 refers to shearing stresses that are mobilized in opposition to both lateral movement and upheaval into the tunnel excavation. The relative magnitude of these two shear stresses depends upon the dimensions involved. In the present case, where the diameter of the tunnel is fairly large with respect to the depth beneath the surface, lateral movement or upheaval probably will be the controlling influence.

Typical computations are given in the Appendix to illustrate the solution of the equilibrium equations and to serve as an example of the accuracy with which these equations reproduce measured pressures. The computations for cases 1 and 2 are for the limiting cases of temporary and final equilibrium, re-

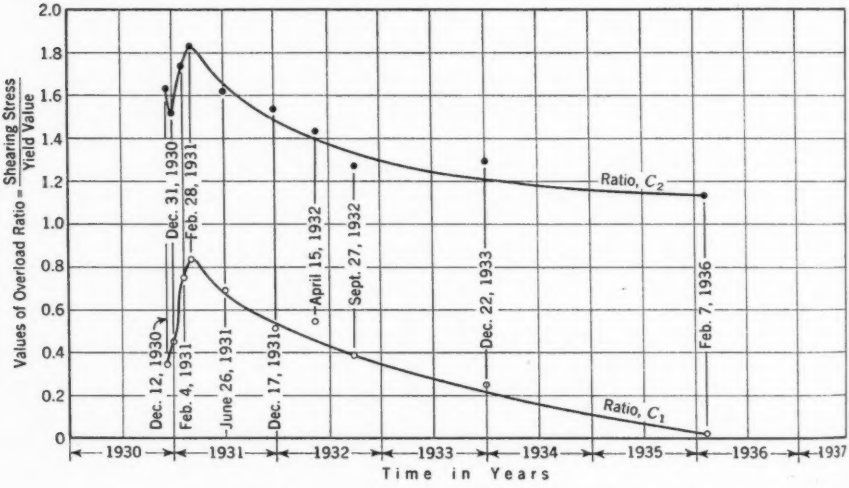


FIG. 45.—VARIATION IN SHEAR STRESS WITH TIME

spectively, and thus represent controlling values of shearing resistance. The range of variation in shear stress is shown in Fig. 45 by the change in C_1 and C_2 with time.

The maximum and minimum values are the controlling factors for practical application. The maximum shear stress is represented by the overload ratio, C_2, equal to 1.84, and occurs at the end of the first stage of pressure variation more than three months after the air pressure is released. From this value of the overload ratio it is concluded that, due to movement of the soil mass toward the tunnel, a shearing stress approaching twice the yield value has been mobilized. A very substantial part of this must be dynamic resistance and

not permanently available. This is borne out by the fact that the value of C_2 decreases almost to unity for final equilibrium, the actual value being 1.12. The final value of the overload ratio is a significant figure as it represents the over-all accuracy with which the proposed method reproduces the observed earth pressure in terms of the independent measurement of shearing resistance. It means, specifically, that for a perfect check the shearing resistance should be 12% greater than the yield value that was actually found by laboratory test. It is impossible to state whether this margin of error is in the shear test or the method of computing the pressure, but it is felt that it is well within practical limits. The values of C_1 are not controlling and they simply indicate the weight transfer away from the top of the tunnel at various stages. The fact that C_1 approaches zero in this particular case indicates that the pressure at the top finally became almost equal to the static head.

Further proof of the validity of the equations used in the circuit method of computing pressure depends on the accuracy with which they reproduce all of the observed pressures. Comparisons of computed and measured pressures for both temporary and final equilibrium are shown in Fig. 46. The computed

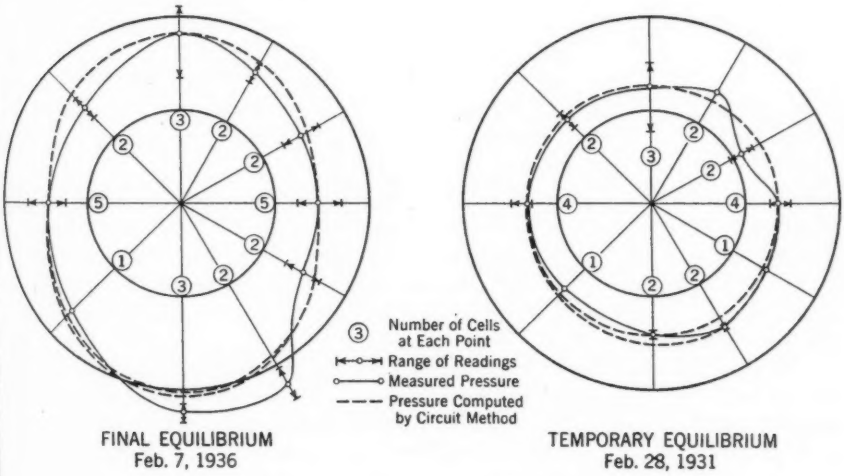


FIG. 46.—COMPARISON OF MEASURED AND COMPUTED PRESSURES

pressures are shown by dashed curves and have been found by substituting the shear-stress values deduced from the previous analysis. Normal pressure, p_N, at intermediate points on the circumference has been found by combining components of the lateral and vertical pressure in accordance with the following equation, in which θ is the angle with the horizontal:

$$p_N = p_h \cos^2 \theta + p_v \sin^2 \theta \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (37)$$

Fig. 46 indicates only that the equations used are capable of reproducing the pressure variations around the entire circumference. The closure of the pressure circuit is reasonably close and indicates that the equations are consistent as a group.

A more important experimental verification is afforded by the comparison shown in Fig. 47. In this case the measured yield value of the soil has been used to obtain the computed values of the earth pressure. The solution is given as case 3 in the Appendix. It is assumed that in final equilibrium the controlling shear values are equal to the measured yield values. Taking a

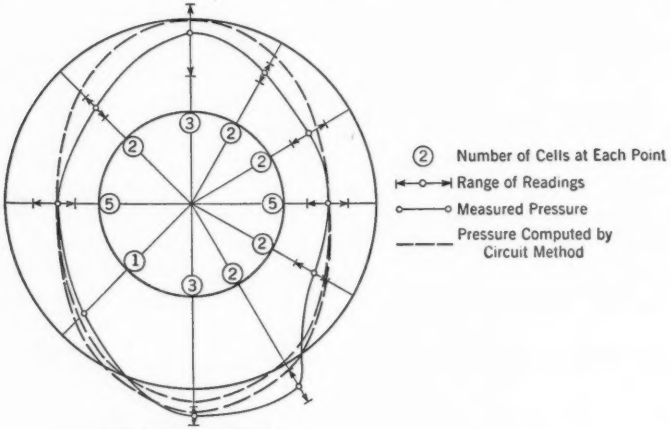


FIG. 47.—FINAL EQUILIBRIUM USING MEASURED SHEAR VALUES

measured value of the lateral pressure as a starting point, the equilibrium equations may be solved for pressures at all other points. The dashed pressure curves shown in Fig. 47 indicate the magnitude of experimental error in terms of measured pressures, corresponding to the 12% error in overload ratio previously mentioned.

Suggested Design Procedure.—In the preceding discussion the earth pressure was known, and direct comparison of measured and computed pressures was possible. In the usual design problem it is presumed that the shearing resistance of the soil, its weight, and the general dimensions of the tunnel will be known and that the earth pressure must be computed. An examination of the equations reveals too many unknowns for a direct solution, and some simplifying condition must be introduced.

One of the most common problems in tunnel construction is that of estimating the air pressure that will be required under a given set of physical conditions. In this case the internal pressure supplied will be equal at all points in the tunnel, and this imposed condition makes the equilibrium equations of the circuit method determinate.

In specifying the shearing resistance to be used in computing the air pressure required, the time element is perhaps the most important factor. It obviously would be unnecessary to use a high enough air pressure to hold shear stresses down to the yield value. In the present observations, an overload ratio of almost 2 represented available resistance over a period of three months. By

allowing the same displacement in a shorter period of time, an even higher overload ratio is possible.

In previous investigations the writer has had a number of opportunities to evaluate overload ratios for progressive settlement of large buildings and has found them to be of the order of 2, or slightly more, without dangerously high rates of settlement. Similar study of embankments and open excavations has indicated a general range of from 2 to 3 for these ratios. Based on these observations, the writer for some time has been using a limiting value of 2.5 for the design of bracing in open excavation or for embankment slopes where some settlement was permissible. Analyses of such designs that have gone through the construction stage appear to indicate that overload ratios approaching or in excess of 3 involve rates of displacement that are excessive and may be approaching the danger zone as far as slides are concerned. It is difficult to supply general design criteria for such a problem, as each individual case should receive special study. Among other factors, the volume of the mass and general dimensions will affect materially the rate of movement produced.

The Central Avenue Tunnel itself supplies the best example of a permissible overload ratio for the specific conditions involved. As noted in the earliest results reported, the tunnel was built under compressed air with a pressure of 27 lb per sq in. It also was found that this pressure held the squeezing to permissible limits but did not completely prevent movement. With this balanced air pressure it is shown in case 4 of the Appendix that the maximum overload ratio was approximately 2.8. Conversely, it may be shown that if the overload ratio for design had been assumed as 2.5, as previously suggested, the required air pressure would have been 30 instead of 27 lb per sq in.

This is a fair example of present practice in tunnel excavation and appears to be quite representative. It seems probable that there are similar examples available from the construction experience of other engineers which would yield valuable design data if subjected to a similar analysis.

Conclusion

The following conclusions should serve to summarize the preceding discussion of earth pressure on tunnels and to emphasize the practical value of shearing resistance tests in evaluating these pressures:

(1) A consistent record of measured earth pressure over a period of ten years indicated that final equilibrium was not achieved until five years after construction.

(2) There were two separate stages of pressure variation. For the first three months after construction pressures were decreasing, indicating a continued mobilization of available soil resistance. Thereafter, vertical pressure increased for several years until static equilibrium was achieved and soil resistance values returned to the yield value as measured by independent shearing resistance tests.

(3) The minimum pressures recorded occurred at the end of the first stage of variation, at which time they were practically the same at all points around the circumference of the tunnel.

(4) At the time of final equilibrium the vertical pressures were about equal to the static head, or even slightly greater, whereas the lateral pressure, which varied but little throughout the observations, was substantially less than the static head.

(5) A method of computing the earth pressure was devised which reproduced the pressures at final equilibrium within practical limits by the use of shear values measured by independent laboratory tests.

(6) Analysis of the temporary condition of equilibrium shortly after construction revealed that dynamic resistance was brought into play with shear stresses approaching twice the yield value.

(7) Analysis of equilibrium involving the use of compressed air indicated that the required shear stresses would approach three times the yield value.

(8) Assumptions were suggested for temporary conditions during construction. These involved an overload ratio of not more than 2.5 for design when some movement and settlement may be permitted.

APPENDIX

Typical Computations to Illustrate the Solution of Equilibrium Equations

Case 1. Temporary Equilibrium.—

$p_v = 5{,}065$ lb per sq ft	$S_v = 300$ lb per sq ft	$H_t = 59.2$ ft
$p_h = 5{,}345$ lb per sq ft	$S_A = 200$ lb per sq ft	$H = 65.6$ ft
$p'_v = 5{,}650$ lb per sq ft	$W_t = 860$ lb per sq ft	$H_b = 72.0$ ft
$\gamma = 125$ lb per cu ft	$W_s = 340$ lb per sq ft	$D = 12.84$ ft

$$p_v = \gamma H_t - \frac{2 S_1 H_t}{D} \quad (S_1 = C_1 S_v)$$

$$5{,}065 = 125 \times 59.2 - \frac{2 \times C_1 \times 300 \times 59.2}{12.84} = 7{,}400 - 2{,}760 C_1$$

$$C_1 = \frac{7{,}400 - 5{,}065}{2{,}760} = 0.84$$

$$p_h = \gamma H + \frac{S_1 H_t}{D} - \frac{S_2 H_t}{D} - 4 S \quad (S_2 = C_2 S_v \text{ and } S = C_2 S_A)$$

$$5{,}345 = 125 \times 65.6 + \frac{0.84 \times 300 \times 59.2}{12.84} - \frac{C_2 \times 300 \times 59.2}{12.84}$$

$$- 4 \times C_2 \times 200 = 8{,}200 + 1{,}160 - 1{,}380 C_2 - 800 C_2$$

$$C_2 = \frac{8{,}200 + 1{,}160 - 5{,}345}{2{,}180} = 1.84$$

$$p'_v = \gamma H_b + \frac{S_1 H_t}{D} - \frac{S_2 H_t}{D} - 5 S - \frac{W_s}{2}$$

$$p'_v = 125 \times 72.0 + \frac{0.84 \times 300 \times 59.2}{12.84} - \frac{1.84 \times 300 \times 59.2}{12.84}$$

$$- 5 \times 1.84 \times 200 - \frac{340}{2} = 9{,}000 + 1{,}160 - 2{,}540 - 1{,}840$$

$$- 170 = 5{,}610 \text{ lb per sq ft} < 5{,}650 \text{ lb per sq ft.}$$

Check:

$$p'_v = p_v + W_t + \frac{W_s}{2} = 5{,}065 + 860 + \frac{340}{2} = 5{,}065 + 860 + 170$$

$$= 6{,}095 \text{ lb per sq ft} > 5{,}650 \text{ lb per sq ft.}$$

Case 2. Final Equilibrium.—

$p_v = 7{,}320$ lb per sq ft	$S_v = 300$ lb per sq ft	$H_t = 59.2$ ft
$p_h = 5{,}805$ lb per sq ft	$S_A = 200$ lb per sq ft	$H = 65.6$ ft
$p'_v = 9{,}120$ lb per sq ft	$W_t = 860$ lb per sq ft	$H_b = 72.0$ ft
$\gamma = 125$ lb per cu ft	$W_s = 340$ lb per sq ft	$D = 12.84$ ft

$$p_v = \gamma H_t - \frac{S_1 H_t}{D} \quad (S_1 = C_1 S_v)$$

$$7{,}320 = 125 \times 59.2 - \frac{2 \times C_1 \times 300 \times 59.2}{12.84} = 7{,}400 - 2{,}760\, C_1$$

$$C_1 = \frac{7{,}400 - 7{,}320}{2{,}600} = 0.03$$

$$p_h = \gamma H + \frac{S_1 H_t}{D} - \frac{S_2 H_t}{D} - 4S \quad (S_2 = C_2 S_v \text{ and } S = C_2 S_A)$$

$$5{,}805 = 125 \times 65.6 + \frac{0.03 \times 300 \times 59.2}{12.84} - \frac{C_2 \times 300 \times 59.2}{12.84}$$

$$- 4 \times C_2 \times 200 = 8{,}200 + 42 - 1{,}380\, C_2 - 800\, C_2$$

$$C_2 = \frac{8{,}200 + 42 - 5{,}805}{2{,}180} = 1.12$$

$$p'_v = \gamma H_b + \frac{S_2 H_t}{D} - \frac{S_3 H_t}{D} - 2S - \frac{W_s}{2}$$

$$p'_v = 125 \times 72.0 + \frac{1.12 \times 300 \times 59.2}{12.84} - \frac{1.12 \times 300 \times 59.2}{12.84}$$

$$- 1.12 \times 200 \times 2 - \frac{340}{2} = 9{,}000 + 1{,}545 - 1{,}545 - 450 - 170$$

$$= 8{,}380 \text{ lb per sq ft} < 9{,}120 \text{ lb per sq ft.}$$

$$p'_v = p_v + W_t + \frac{W_s}{2} = 7{,}320 + 860 + \frac{340}{2} = 7{,}320 + 860 + 170$$

$$= 8{,}350 \text{ lb per sq ft} < 9{,}120 \text{ lb per sq ft.}$$

Case 3. Final Equilibrium Using Measured Shear Values.—

$p_h = 5{,}805$ lb per sq ft	$C_2 = 1$
$S_v = 300$ lb per sq ft	$S_2 = S_3 = S_v$
$S_A = 200$ lb per sq ft	$S = S_A$

$$p_h = \gamma H + \frac{S_1 H_t}{D} - \frac{S_2 H_t}{D} - 4 S$$

$$5{,}805 = 125 \times 65.6 + \frac{S_1 \times 59.2}{12.84} - \frac{300 \times 59.2}{12.84} - 4 \times 200$$

$$S_1 = \frac{5{,}805 + 1{,}380 + 800 - 8{,}200}{4.61} = -47 \text{ lb per sq ft.}$$

$$p_v = \gamma H_t - \frac{2 S_1 H_t}{D} = 125 \times 59.2 - \frac{2 \times (-47) \times 59.2}{12.84}$$

$$= 7{,}400 + 435 = 7{,}835 \text{ lb per sq ft} > 7{,}320$$

$$p'_v = \gamma H_b + \frac{S_2 H_t}{D} - \frac{S_3 H_t}{D} - 2 S - \frac{W_s}{2}$$

$$p'_v = 125 \times 72 - 400 - 170 = 9{,}000 - 370 = 8{,}430 \text{ lb per sq ft} > 9{,}120.$$

Check:

$$p'_v = p_v + W_t + \frac{W_s}{2} = 7{,}835 + 860 + 170 = 8{,}865 \text{ lb per sq ft} < 9{,}120.$$

Case 4. Temporary Equilibrium Under Balanced Air Pressure.—With a balanced air pressure of 27 lb per sq in. in the tunnel, find various overload ratios.

27 lb per sq in. = 3,888 lb per sq ft = $p_v = p_h = p'_v$

$S_v = 300$ lb per sq ft. $S_A = 200$ lb per sq ft.

Temporary Equilibrium.—

$$p_v = \gamma H_t - \frac{2 S_1 H_t}{D}$$

$$3{,}888 = 125 \times 59.2 - \frac{2 \times C_1 \times 300 \times 59.2}{12.84} = 7{,}400 - 2{,}760 C_1$$

$$C_1 = \frac{7{,}400 - 3{,}888}{2{,}760} = 1.27$$

$$p_h = \gamma H + \frac{S_1 H_t}{D} - \frac{S_2 H_t}{D} - 4 S$$

$$3{,}888 = 125 \times 65.6 + \frac{1.27 \times 300 \times 59.2}{12.84} - \frac{C_2 \times 300 \times 59.2}{12.84}$$

$$- 4 \times 200 \times C_2 = 8{,}200 + 1{,}750 - 1{,}380 C_2 - 800 C_2$$

$$C_2 = \frac{9{,}950 - 3{,}888}{2{,}180} = 2.78$$

$$p'_v = \gamma H_b + \frac{S_1 H_t}{D} - \frac{S_2 H_t}{D} - 5 S - \frac{W_s}{2}$$

$$3{,}888 = 125 \times 72.0 + \frac{1.27 \times 300 \times 59.2}{12.84} - \frac{C_2 \times 300 \times 59.2}{12.84}$$

$$- (5 \times C_2 \times 200) - 170 = 9{,}000 + 1{,}750 - 1{,}380 C_2 - 1{,}000 C_2 - 170$$

$$C_2 = \frac{6{,}692}{2{,}380} = 2.81.$$

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

REPORTS

PROTECTIVE AND REMEDIAL MEASURES FOR SANITARY AND PUBLIC HEALTH ENGINEERING SERVICES

SECOND PROGRESS REPORT OF THE SANITARY AND PUBLIC HEALTH ENGINEERING DIVISION OF THE NATIONAL COMMITTEE OF THE SOCIETY, ON CIVILIAN PROTECTION IN WAR TIME

1. General Statement

The National Committee of the Society on Civilian Protection in War Time was first thought of at the Annual Convention in Denver, Colo., in 1940. The Committee took form in the late fall of that year and presented its first review of civilian protection at the Annual Meeting of the Society in January, 1941. At that time war was regarded as very remote and general interest in civilian protection was quite limited. The formation of the Committee was a foresighted action by the Society. The first progress report of the Division on Sanitary and Public Health Engineering was published in *Proceedings* for January, 1942, one month after the United States entered the war; but it was prepared and largely completed prior to December, 1941.

Now, it is pertinent to take stock of the objectives of the Committee and the development of the technology of civilian protection. Much has been accomplished and many states and municipalities have progressed far in preparing for civilian protection. Those who have been coordinating and arranging the data thus created are grateful for the opportunity to take part in this program for civilian protection in wartime in the engineering field.

To appraise the development of this technique the writer has set down the following steps or stages:

(*a*) Preliminary study of experience;
(*b*) Preparation of general plans of procedure;

Note.—This Progress Report is the work of Samuel A. Greeley, M. Am. Soc. C. E., Chairman of the Sanitary and Public Health Division, with the assistance of many Local Section Chairmen throughout the United States. Please forward all comments on this Report directly to Chairman Greeley, (Greeley & Hansen), 6 North Michigan Ave., Chicago, Ill. Progress Reports are published in *Proceedings* only.

(*c*) Perfecting local organizations for putting plans and procedures into effect;

(*d*) Training personnel and practicing under assumed emergency conditions;

(*e*) Making detailed surveys and construction drawings for necessary protective structures;

(*f*) Acquiring material and equipment; and

(*g*) Building protective structures.

The information that has been received by the Committee indicates great progress through preliminary study, general plans and procedures, organization, and, to a limited extent, the training of personnel. Some progress has been made in the acquisition of material and equipment and to a less extent in the actual building of protective structures. When the writer began preparing a report to the Society at the Spring Meeting in Roanoke, Va., he expected that it might be limited to the design and construction of protective works; but after receiving many returns from the Local Sections, he concluded that the report should refer also to the various stages of civilian protection that begin with and follow actual organization. It is important also that the matter of appropriations for actual construction be considered.

2. The Question of Appropriations

The first progress report of the Committee included a paragraph entitled "Funds and Budgets." No actual expenditures for construction were recommended. In some localities appropriations have been made and in others may be much needed. It is possible that some communities and authorities are not yet ready to make expenditures or may not need to make them. The opportunity must be provided, however, to evaluate such engineering structures and their cost in the light of their need for civilian protection under war conditions. An object of this report, therefore, is to throw some light on the question of general policy regarding this matter.

3. Insurance Aspects

The first progress report called attention to some of the general aspects of sanitary and public health engineering such as street cleaning, public comfort stations, milk supplies, rooming houses, and restaurants. These aspects do not have as direct or as spectacular an effect on civilian protection as do the public health services of water supply, sewerage, and refuse disposal. In many localities, however, they may have a far-reaching adverse effect on public health and morale. Expenditures for civilian protection in such general facilities are in the nature of an insurance and perhaps should be appraised in relation to total war expenditures including military, naval, and civilian.

4. Construction Measures

In a broad sense, the term "construction measures" is intended to include any procedure for civilian protection which involves expense. Among the obvious and preliminary items are fencing, flood lighting, and arrangements

for blackouts. Based in part on British experience, the Committee has listed a number of construction measures as follows:

(*a*) Reinforcing the water distribution system by new cross connections and valves;

(*b*) Providing emergency sources of water supply by connecting with nearby communities and other local supplies;

(*c*) Providing emergency pumping and generating equipment;

(*d*) Building protective covers over important underground structures;

(*e*) Providing bomb-proof vaults for records;

(*f*) Constructing splinter and blast protecting walls around important pieces of equipment;

(*g*) Providing floating and other equipment for the protection and repair of dams;

(*h*) Building or arranging for emergency intake works at some distance from the regular intakes;

(*i*) Taking special precautions regarding the sanitation of public comfort stations, milk supplies, rooming houses, and restaurants;

(*j*) Providing alternate arrangements for garbage and refuse disposal;

(*k*) Acquiring the necessary tools, repair parts, equipment, and supplies for making emergency repairs and for maintaining the operation of sewers, sewage pumping stations, and treatment plants;

(*l*) Acquiring mobile equipment for water purification and treatment and for power production; and

(*m*) Placing underground or duplicating important conduits.

Later paragraphs in this report describe some of the progress that has been made in the latter stages of the development of engineering technology for civilian protection in wartime in the field of sanitary and public health engineering. In a country so large and diversified as the United States, standards will differ and relative values should be considered. The most important and vulnerable communities would seem to require the largest expenditures. As some of the communities have asked that the information regarding them be stated so as not specifically to identify the locality, the writer has refrained from giving the names of the communities in some cases.

5. Chicago Civilian Defense Plan

One of the most interesting and well conceived organizations for civilian protection has been developed in Chicago, Ill., by the Commissioner of Public Works, Oscar E. Hewitt, and the City Engineer, W. W. DeBerard, M. Am. Soc. C. E. This plan has been arranged in a series of reports, each dealing with a hypothetical but specific emergency problem, such as the repair of water mains, of pumping stations, of sewers, of the airport, and of bridges.

The water-pipe extension division has already had experience with breaks in water mains including the repair of six breaks in large mains. This experience is the basis of the plan. Although these six breaks occurred over a period of years, the emergency plan is based on making them all during a single emergency. A canvass of the existing facilities showed that the division could

handle six major water main breaks simultaneously and restore water service completely within about 24 hr. The report itemizes the needed additional equipment estimated to cost about $350,000 which would enable the division to handle not six but eight major water main breaks and to reduce the average time of repair for all of the eight breaks to 16 hr. The report describes in detail the procedure to be followed in handling these six major breaks, including the personnel and a time table of their appearance. To illustrate, the following is quoted from this report:

> "The following is a report of damage assumed to have been done to water system by Bombing Raid on February 3rd, 1942. There were two major breaks reported in the North District at 1: A.M., one located on 36 inch main in North Avenue at Tripp Avenue and the other on the 30 inch main in Clybourn Avenue 300 feet North of Cortland Street.
>
> "A Foreman located in North District #1 at Clybourn Avenue and Webster Avenue, immediately dispatched two assistant foremen, four calkers, and twelve laborers to the site of the break located at Clybourn Avenue 200 feet North of Cortland Street and five minutes later began the work of making the shut down on Clybourn Avenue from Cortland Street to Fullerton Avenue, which involved a 30 inch valve at the South line of Cortland Street, a 24 inch valve at the North line of Fullerton Avenue and six smaller grid system valves adjoining. During the shutdown, the equipment consisting of an emergency trailer, two pumps and two air compressors arrived at the site and were spotted by an assistant foreman. An emergency truck crew consisting of chauffeur, junior engineer and two laborers arrived at 1:10 A.M., and assisted the district crew in making the shutdown. Then, in the next ten minutes, three additional assistant foremen, four calkers, four plumbers and 34 laborers arrived at the site and were immediately dispatched to locate and open sewer manholes and other inlets to permit drainage of flood waters. The shutdown was completed in one hour and 15 minutes or at 2:20 A.M. Four pumps were started and in three hours or at 5:20 A.M. the opening was dewatered and the nature of the damage was viewed. There was a crater 50 feet in diameter and 30 feet deep and the entire section of the 30 inch main, together with a 30 inch sewer was destroyed.
>
> "While the above operations were proceeding, the junior engineer, who made a survey of the situation, contacted an assistant engineer at the North District headquarters, who ordered the engineering party to make a pressure survey of the area from Division Street to Diversey Parkway and from Damen Avenue to Sedgwick Street and report back on same.
>
> "After the crater was dewatered to the level of the 30 inch sewer, it was necessary to temporarily dam off the sewer on either side to permit further pumping, after consultation with the Bureau of Sewers. The balance of the water was then pumped out. It was then planned that the quickest and cheapest method of handling the situation would require a run around the circumference of the crater. Accordingly, two assistant foremen supervising 4 calkers and 30 laborers, extended the opening on either side of the crater and excavated a shelf around the circumference of the crater westerly toward the property line to support the proposed rerouted pipe. The air compressors were utilized for this work. It was then determined that it would require 10 lengths of 30 inch pipe, four 30 inch ⅛ Bends and two 30 inch sleeves, which was then promptly ordered from Pipe Yard 'B' at 31st Street and Sacramento Avenue. The foreman checked over the available gang and dismissed the surplus men, after assigning them to certain relief shifts.

"While the district forces were so occupied, the junior engineer contacted the assistant engineer and reported pressures generally satisfactory and also requested the installation of two additional taps for purposes of chlorination and sampling of water. A tapper was dispatched to the engineer who furnished him with the location of the requested taps.

"Lack of water complaints were traced in cooperation with Plumbing inspectors to blockage of service pipes and meters with dirt, resulting from change of direction of flow in the grid system, due to the shutdown. Two plumbers were dispatched with force pumps to clear service pipes and two plumbers to clean meters.

"In the interval, the foreman ordered truck crane to be sent from break at North and Tripp Ave. at 11:20 P.M., February 4th, 1942, the work of installing the 120 feet of main together with four 30 inch ⅛ bends commended. This operation was completed at 12:20 A.M. February 5th, 1942. The main was then filled with water by opening the 30 inch valve at the South line of Cortland Street and the Chlorinator installed at this point. The main was flushed and chlorinated continuously at the direction of the Water Safety Control Division until satisfactory for drinking purposes. Upon receiving this approval from the Water Safety Control Division, the remaining valves in the closed section were opened and the main restored to service. The main was ready for service at 2:05 P.M. February 5th, 1942, or 49 hours and five minutes after first reported. The trench was not backfilled by the Water Pipe Extension forces in order to permit other utilities to make necessary repairs to their equipment and who had agreed to complete the backfilling.

"After the main had been shut down the Damage Investigators assisted by 12 men drawn from the office personnel, investigated the location and extent of damage to private property."

On April 21st, there was an actual practice mobilization of the personnel, equipment, and materials for these hypothetical breaks.

6. Seaboard City (A)

This relatively large municipal utility district is employing eighty-four guards at a monthly cost of about $15,000 to protect dams, reservoirs, aqueducts, water treatment plants, and pumping stations. The actual construction work that has been undertaken includes the placing of certain exposed transmission and distribution lines underground, erection of fences and flood lights, and in addition the acquisition of a portable gasoline engine generator and two portable gasoline engine pumps.

7. Midwest Industrial City (B)

This community has made very substantial progress in providing for civilian protection. Already some $2,000,000 has been made available for construction work, and a project estimated at $1,500,000 is under consideration for additional capacity and stand-by equipment at water pumping stations. The largest item in the projects already financed is for the reinforcement of the distribution system; but there is also included money for fencing, flood lighting, emergency repair materials and supplies and for much miscellaneous equipment, such as guard houses, guard equipment, and the like. Throughout the city a number of small surface reservoirs have been established to provide water for local fire protection in the event of emergency disruption of the distribution system.

8. New York Metropolitan Area

The reports indicate that considerable is being done in the communities adjacent to New York, N. Y. Jersey City, N. J., for instance, has undertaken the following measures for civilian protection:

(*a*) The distribution system has been reinforced by additional control valves at strategic locations.

(*b*) Two additional large interconnections with adjacent water supplies will be made in the near future.

(*c*) An emergency gasoline engine generator has been installed.

(*d*) Manhole covers on important supply lines over valves have been fastened down to prevent them from being opened by people other than regular employees.

(*e*) Steel doors and steel shutters for windows have been built on brick gate houses.

(*f*) The city has arranged for alternate dumping grounds, for garbage disposal, and for additional trucks for the extra haul.

In Suffolk County, New York, steps are being taken for the building of connections to adjacent water supplies. In Newark, N. J., a 60-in. steel pipe more than one mile long is under construction to provide an alternate supply of water to the city.

9. West Coast City (C)

This city has under way a project, estimated at a cost of $445,000, for reinforcement of the distribution system including new cross connections and valves. The city has acquired gasoline engine pumps, and has arranged for tank trucks to carry drinking water to areas temporarily without water because of a broken main. The report from this community states that a great effort has been made to enable the department to isolate quickly any bombed area and to confine that area to the smallest possible extent.

10. Midwest City (D)

This city has recently completed a new water supply line from a large lake which includes an intake structure and low lift pumping station on the lake shore, and a pipe line about 65,000 ft long to the city. It has been recommended that the manholes along the pipe line be covered with earth to make them less conspicuous and less accessible to saboteurs. The old water works, steam operated, are being superseded by the new supply, are in good operating condition, and can be placed in operation in about 5 hr. As a war emergency, the old water works will be kept in service. It is proposed to install either a gasoline engine pump or a motor driven pump in an old Steam Station so as to have some pumping equipment that can be started quickly. Such a pump will draw from two existing reservoirs which will be kept full. By starting the pumps, time will be provided for starting up the old steam plant in the event of a serious break in the new system. At the sewage treatment

plant, gates have been installed on the approaches to discourage ordinary sabotage.

11. Ohio Cities

The report of the local Chairmen indicates that steps have been taken to provide water connections between adjacent communities as at Akron, Barberton and Cuyahoga Falls, and at Painesville and Fairport. Cleveland has spent about $42,000 on civilian protection and is considering the appropriation of an additional $40,000. The City of Cleveland has provided fifty-six emergency watchmen at an annual cost of more than $80,000.

12. Texas

Texas has been active through its State Sanitary Engineer. It has prepared an organization of an emergency sanitary corps and has given consideration to the following activities:

(*a*) Program to prevent overcrowded and insanitary housing conditions;
(*b*) Protection of water supplies and of sewage and garbage disposal facilities;
(*c*) Insect and rodent control; and
(*d*) Rural sanitation.

Emergency chlorinating equipment is available through the State Board of Health.

13. Phoenix, Ariz.

It has been reported that the City of Phoenix has rehabilitated emergency or standby wells which have been held in readiness for service in case of an interruption in the normal water supply and it has recently arranged to borrow five chlorinators for installation at the emergency wells.

14. Southern City (E)

This city has a well-water supply. It has just completed a survey of some thirty privately owned deep wells scattered throughout the city which it plans to equip with fire motor connections so that they will be available for fire protection through the fire department, should there be a local failure of the general water supply system. In addition, fences have been installed around important plants, manhole covers have been equipped with locks, and an underground Diesel driven generator is being installed to serve as an emergency source of power for the municipal radio station.

15. Repair of Sewers

A plan prepared by the Bureau of Sewers of the City of Chicago gives a vivid picture of the procedure for the repair of broken sewers. After careful investigation it was concluded that the existing force and the available equipment and materials could repair, at one time, eleven sewer breaks. It was estimated that the broken sewers could be made usable within 6 to 8 hr and that complete reconstruction would require from 3 to 10 days. The hypo-

thetical broken sewers included sizes from 15 in. to 9.0 ft in diameter and lengths from 38 to 800 ft. The report states that British experience indicates that eleven sewer breaks is an extremely small number, and that three or four times this number could be expected during a bombing raid. Preparation to handle twenty-two breaks at one time would require additional equipment and materials estimated to cost about $400,000. The organization of the repair procedure has been carefully worked out and will shortly be given an actual try out for an assumed break. A typical procedure is described as follows:

> "Case 1. A bomb probably aimed at the Lake View pumping station or the Uptown L Station blows up a 38 ft. section of 36 inch sewer 11½ ft. deep in Lawrence Ave. between Clarendon Ave. and Sheridan Rd. at 11 p.m. together with gas and water mains. Edison and Telephone conduits, street lighting system, pavements, walks, and parts of four buildings which are afire and burning fiercely because of escaping gas. The alarm from both police and fire departments is taken at the office of the Bureau of Sewers by men of the 4 to 12 p.m. detail. These men call up the foremen and employees of Squad 'A' (shown in tables of Emergency Sewer Squads), who report promptly at the site of the explosion. The first men of this gang arrive 25 minutes after the alarm reaches the Bureau of Sewers, having averaged only ½ to 1 mile travel.
>
> "Fire and police trucks from near stations have arrived. Sewer workmen after identifying themselves help form a cordon to keep people back until the Water Pipe Extension crew can shut valves in water mains and Gas Co. gangs can stop the flow from a broken gas main. Meanwhile, the firemen are getting fires under control, first in the crater in the street and then in the buildings. The Supt. of Sewers has ordered out trucks from the yard at 4100 No. Damen Ave. 2½ miles away, with lumber for sheeting and bracing, brick and cement for rebuilding sewer and a manhole and catchbasins as well as a portable electric light outfit, pumps, hose and connections, and a full outfit of tools. The sewer gang clears debris so as to install pumps to unwater the crater and handle the flow of the sewer. The air compressor is set up and air tools are breaking up large slabs of concrete in the crater while other sewer workmen are clearing debris or hauling it to temporary dumps in side streets. Trucks are being loaded by a speed crane set in a cleared area beside the crater and excavating muck and debris from the crater so that workmen can get at the sewer and other underground structures. After excavation has progressed sufficiently, the sewer and appurtenances are rebuilt, all local connections repaired, the crater backfilled, and the street restored to traffic. Four days non-continuous work will complete sewer repair at this spot."

16. Camouflage

No information regarding practice in camouflage has been received by the Committee, but there may be instances where it would prove useful, as for instance the right method of painting exposed structures. It is understood that a report on this subject will soon be sent to the Local Section Chairmen.

17. State Mutual Aid Plans

Mutual aid plans have been developed in a number of states. These have been directed in the first instance to mutual aid for the maintenance of water supply service. One of the most completely developed mutual aid plans is that of the State of New York. To illustrate what has been accomplished, the

following is abstracted from the Mutual Aid Plan Progress Report as of March 1, 1942:

(*a*) Interconnections between water systems—
Number proposed 236
Number for which plans have been prepared 141
Number installed 196
(*b*) Maps of distribution systems—
Number submitted 296
(*c*) Surveys of emergency water supplies—
Number completed 257
(*d*) Surveys for weaknesses in distribution system—
Number completed 59
(*e*) Special investigations of water service and fire protection to war industries—
Number completed 35
(*f*) Special studies of power supplies—
Number completed 12
(*g*) Active training of auxiliary water-works personnel—
Number of communities 23
(*h*) Inventories of personnel, equipment, materials, and supplies—
Number prepared and submitted 613

The Committee in charge of the mutual aid plan has prepared an act which gives municipalities additional powers during the present emergency to make interconnections and otherwise participate in the mutual aid plan. An inventory has been prepared of the available supplies of liquid chlorine throughout the state giving their location. Under the plan the state is divided into twenty-three zones and includes the following groups for its administration: Mutual Aid Committee, Advisory Committee, Cooperating Committees, zone coordinators, state coordinator, and governor of the state.

Among the first duties given to the coordinators were the following:

(*a*) Work out plans for the interconnection of water supplies in each zone and adjoining zones.

(*b*) Study the necessary reinforcement of the distribution system.

(*c*) Make a detailed inventory of personnel, material, equipment, and supplies in each zone and arrange for prompt interchange.

Mutual aid plans of this kind have been set up in other states, sometimes under the same name and sometimes as state emergency organizations, among which are North Carolina, Massachusetts, Texas, Georgia, Illinois, Maryland, Indiana, etc.

18. Indiana Emergency Water and Sewerage Committee

The Indiana Committee is directed and sponsored by the State Board of Health and the Indiana Defense Council. It has issued some very good bulletins. One of these, *Bulletin No. 1*, dated January 30, 1942, includes thirty-six questions and answers. One of the questions, for instance, is "What

are to be the purposes and duties of the Committee?" The answer states that the members of the Committee are to go into any municipality to assist in the development of auxiliary water supplies and the restoration of water and sewerage service in an emergency.

Another question is as follows: "We can't afford to carry a large supply of emergency distribution repair fittings and we have no special tools. Will the Committee get them for us in an emergency?" The answer is that an inventory was being made of the available supplies and equipment in every water works of the state and that a mutual aid plan would be developed on the basis of this inventory.

It is also stated in reply to a question that the emergency Committee has five gas chlorinators and three hypo-chlorinators available.

Much information has been prepared and distributed regarding the general aspects of civilian protection as, for instance, the statement that dams and intakes should be protected against floating explosives by installing skimmers (and nets?) upstream from the face of the structures. It was of interest to note a special paragraph under General Information as follows:

> Keep yourself physically fit and impress upon your employees the need for being physically fit to meet any conditions, long hours, exposure, change of food, that might be prevalent during an emergency.

19. Florida Mutual Aid Plan

A memo dated January 5, 1942, from the Florida Bureau of Sanitary Engineering includes the following topics:

(*a*) Instruction of employees as to detailed operation of the plan;
(*b*) General outline of mutual aid plan;
(*c*) Possible interconnections with contiguous municipalities, and approved industrial water supplies;
(*d*) Maps of distribution system;
(*e*) Cooperation with fire departments;
(*f*) Detailed survey of all possible sources of water supply;
(*g*) Water service for war industries;
(*h*) Cooperative work with other utilities;
(*i*) Training auxiliary employees to make repairs;
(*j*) Precautions against sabotage; and
(*k*) Inventory of water-works personnel, equipment, etc.

20. Miscellaneous Items

Under the Georgia emergency protection program, the State Department of Public Health has acquired chlorinating equipment and a portable water laboratory.

The Delaware Division of Sanitary Engineering has placed two men on restaurant inspections in important war areas in Kent and Sussex counties.

Under the direction of the Bureau of Sanitary Engineering in Maryland, emergency cooperative water-works districts have been established. Inventories of supplies, material, and equipment have been made. Some inter-

connections between adjacent distribution systems have been built and others planned.

The City of Detroit has built splinter and blast protecting walls around important pieces of equipment and others are being constructed around electric motors, generators, and switchboards. Exposed windows in rooms housing important pieces of equipment are being filled in with bricks.

The State Sanitary Engineer of the Board of Health of Oregon has assisted in the preparation of a public health preparedness program addressed to County Health Officers and including sections relating to water supply, nursing, first aid, volunteer service, war gases, communicable disease control, emergency chlorination, and directions for sterilization of water.

One of the replies suggested that swimming pools be arranged to serve as emergency reservoirs for fire protection.

21. Repair Crews and Repair Inventories

The following data have been prepared by the New York State Office of Civilian Protection through the Division of Water Main Emergency Repairs and under the direction of Earl Devendorf, Assoc. M. Am. Soc. C. E., Director.

> "In England, under bombing attacks, water distribution systems have suffered breaks in a ratio as high as 1 per mile of pipe. What may occur in American experience is not predictable but it is hardly imaginable that bombings of American cities will be as severe as those which have occurred in England. It is suggested that as a start the number of emergency repair crews which should be organized, developed and trained for communities in or within 10 miles of likely target areas be based on the following table which indicates the number of emergency water main repair crews needed for municipalities of stated population ranges:

Population ranges	No. of emergency repair crews needed
less than 2,000	1
2,000 to 5,000	2
5,000 to 9,000	3
9,000 to 14,000	4
14,000 to 20,000	5
20,000 to 28,000	6
28,000 to 34,000	7
34,000 to 42,000	8
42,000 to 49,000	9
49,000 to 75,000	10 to 12
75,000 to 100,000	12 to 15
100,000 to 200,000	15 to 24
200,000 to 300,000	24 to 32
300,000 to 400,000	32 to 40
400,000 to 500,000	40 to 48
500,000 to 600,000	48 to 54

"In the case of municipalities which are considered to be from 10 to 20 miles of likely target areas in order to create a nearby reservoir of emergency crews which can be quickly dispatched to stricken areas emergency crews in the ratio of about 50% of those indicated above should be organized.

"In the isolated communities which are considered to be well removed from possible target areas the need for organization of emergency crews will be much lessened. But each such community should proceed to have at least one emergency crew organized and trained."

22. Indicated Design and Construction Procedure

The foregoing review of information and experience from many parts of the United States indicates the following procedure in that stage of civilian protection which requires expenditure of funds, the preparation of plans, and actual construction:

(*a*) Surveys of existing facilities;

(*b*) Report on needed additions to meet war emergencies in the light of local conditions;

(*c*) Items indicated by present undertakings as being important: Reinforcing distribution system, interconnections, emergency portable equipment, fencing and lighting, stocks for maintenance and repairs, splinter walls, protective coverings, auxiliary sources of water, small surface storage reservoirs (swimming pools), and alternate arrangements for garbage disposal.

This list indicates the steps that may be included in those stages of the technology of civilian protection which follow the training of personnel and practicing under assumed emergency conditions.

23. Summary Statement

Two broad aspects of civilian protection in sanitary and public health engineering can be described in summary. The first embraces those stages of the technology which depend mainly on the use of existing facilities and personnel and of volunteer effort. These stages comprise preliminary studies, plans of procedure, organization, and actual field practice under assumed emergency conditions.

The second embraces those stages which require expenditures of funds appropriated for the purpose, such as engineering investigations, detailed plans, and actual construction. The need for such expenditures must be established and should be related to a general policy regarding vulnerability, likelihood and extent of damage, and the insurance value of the expenditure. The suggestion is pertinent that such a national policy should be outlined so that the effort and money may be coordinated with the entire war effort. The problem is illustrated by a specific example of an hypothetical case.

Assume an important pumping station comprising three main rooms—one housing the boiler plant, and the others containing groups of old standby steam pumps and of new motor driven centrifugal pumps, respectively. Procedures may have been planned and actually practiced for repairing the damage

caused by a direct bomb hit in one of the pump rooms on the assumption that some service could be restored quickly by placing the other pump room in operation. The effect of splinters and blast on the unhit pumps may have been discounted. Should the department build splinter and blast protection walls around these important pumping units to isolate and limit the damage and, if so, to what extent?

Obviously this should not be done in all places; but may well be done in others. The same question arises for all the other problems of construction expense related to civilian protection.

The needed policy should clarify such matters as what areas are in danger zones or zones of expectancy and what centers within these zones are more important than others. On some such basis the extent of construction work to be undertaken should be oriented to a nation-wide policy of civilian defense effort including cost. Then the engineer should plan a sound procedure for each situation. For preliminary discussion some zones of expectancy might be listed as follows:

(*a*) Washington, D. C.
(*b*) Canal Zone
(*c*) Major seaports: New York, N. Y., Los Angeles, Calif., San Diego, Calif., Hampton Roads, Va., Philadelphia, Pa., Houston, Tex.
(*d*) Important industrial centers within, say, 150 miles of the Atlantic and Pacific Coasts.
(*e*) Large Inland cities having war industries
(*f*) Military and naval establishments depending on civilian service

The writer wishes to call attention particularly to the importance in the field of public health engineering of general sanitation problems relating to public comfort stations, milk supplies, rooming houses, and restaurants. One of the major possible damages to the war effort is the spread of contagious disease through unclean utensils in restaurants and homes. The sanitation problems at the receiving end of evacuations are important in this regard.

Acknowledgment

The Chairman wishes to thank those who have worked so hard and contributed so much information for the use of the Committee. In many cases individual acknowledgment has not been made because of lack of time; but such acknowledgment is none the less intended. It is hoped that much discussion will permit the final completion of a third progress report that will be of some use to those charged in each locality with the responsibility for civilian protection in wartime.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

REPORTS

ADVANCEMENT OF SANITARY ENGINEERING

REPORT OF THE COMMITTEE OF THE SANITARY ENGINEERING DIVISION ON ADVANCEMENT OF SANITARY ENGINEERING

Appointment and Organization of the Committee

This Committee was first established, by action of the Executive Committee of the Sanitary Engineering Division, on January 16, 1934. It was then designated "The Committee on the Promotion of the Status of Sanitary Engineering." On January 14, 1935, the Committee was given its present title.

During 1934 the Committee personnel consisted of the five members of the Executive Committee of the Sanitary Engineering Division plus one member selected from the Division at large. During 1935 the Committee was composed of the Division's Executive Committee of five with three additional members.

On January 14, 1936, the Committee was reorganized by the Executive Committee as a standing committee of the Division with a membership independent of that of the Executive Committee. During 1936 and 1937 the Committee had a membership of five. It had seven members during the period of three years, 1938–1940, and eight members during 1941.

Functions of the Committee

The purpose of this Committee has been stated[1] as follows: "* * * to stimulate the participation of sanitary engineers in the development of public health projects." This compressed statement of function is suggestive of broad possibilities and is therefore intriguing. It is doubtless reasonably satisfactory for the purposes of the *Yearbook*. However, a more elaborate and definite statement of function is desirable if the best efforts of the Committee are themselves to be stimulated and if, therefore, the most effective results are to be secured.

Holding the various possibilities in review it would appear that the Committee might function:

(1) Simply as an historical body, to record the general progress and achievements in the field of sanitary engineering;

Note.—This Progress Report was presented at the meeting of the Sanitary Engineering Division, New York, N. Y., on January 21, 1942. Please forward all comments on this Report directly to Chairman Charles Gilman Hyde, Room 11, Eng. Bldg., Univ. of California, Berkeley, Calif.

[1] *Yearbook*, Am. Soc. C. E., 1942, p. 16.

(2) As a purely educational group, serving to publicize the work of the sanitary engineer and thereby promote his interests;

(3) As an appraising body, attempting to survey the possible field of service of the sanitary engineer and to extend the boundaries of his activities to encompass all logical opportunities for which he is or should be fitted by reason of his training and experience;

(4) As a group charged with the duty of devoting a proper amount of attention and energy to the advancement of the welfare of the public, in so far as this is related to the work of the sanitary engineer considered in its broadest aspects, by:

(*a*) Encouraging the engineering profession to accept, to sponsor, and to protect from exploitation the rapidly developing specialty or specialized field of so-called public health engineering;

(*b*) By improving the quality of the personnel practicing sanitary engineering, through better education and training;

(*c*) By advocating the employment, whenever available, of adequately trained and experienced personnel, only, where such training and/or experience are desirable; and

(*d*) By informing laymen and engineers in other branches of the profession concerning the public health implications of their work.

This Committee believes that it will best serve the interests of the engineering profession, of the public, and of the sanitary engineer himself, if it shall undertake all of these functions, particularly those outlined under categories (2), (3), and (4). It holds the opinion that a long-term program should be developed looking to the broadest and most useful field of service reasonably possible to sanitary engineers.

Among the ends to be diligently sought is a better general appreciation of the potential and actual service that the sanitary engineer can render, and of the compensation which should be considered reasonable and adequate.

Activities of the Committee

During its first two years of existence, the Executive Committee of the Sanitary Engineering Division, with first one (1934) and then three (1935) additional members selected from the Division at large, sat as The Committee on the Promotion of the Status of Sanitary Engineering at each of its annual meetings in New York, N. Y. The new Committee, as reorganized by the Executive Committee at its meeting on January 14, 1936, held an all-day session in New York City on January 19, 1937. The 1940 Committee held sessions on October 16 and 17, 1940, at Cincinnati, Ohio, at the time of the Fall Meeting of the Society.

In 1934 the Committee, through its Chairman, conducted an inquiry to determine the numbers of trained public health or sanitary engineers in the health departments of states, counties or districts, and cities in the United States.[2]

[2] "The Trained Public Health Engineer in Public Health Departments," *American Journal of Public Health*, Vol. 26, No. 7, July, 1936, pp. 697–710.

Reports of This Committee

At a session of the Division of Sanitary Engineering on January 17, 1935, at the Society's Annual Meeting in New York City, the Chairman gave a verbal report stating the functions of the Committee as they were then conceived, outlining some of the problems before it, and describing the activities of the Committee during its first year of existence.[3] Under date of January 19, 1937, the Committee presented what it designated its First Progress Report to the Executive Committee of the Division. It was read at a Division session held on January 21, at the Annual Meeting of the Society. This Report was fairly comprehensive. It was mimeographed and copies were supplied to Society headquarters, to the members of the Committee, and to various other interested persons.

An informal Progress Report was presented orally by the Chairman to the Sanitary Engineering Division at its session on January 17, 1940, at the time of the Annual Meeting of the Society in New York City. The purpose and work of the Committee were described briefly and discussion was requested on three matters:

(1) The relative advantages of the title sanitary vs. public health engineer;

(2) The extension of the Sanitary Engineering Division to include Associates who might not be eligible to corporate membership in the Society, but who are concerned with industrial hygiene and other operations in the control of man's environment, with the hope of forestalling other competitive national organizations of such persons; and

(3) The effect of local civil service regulations upon the best interests of technical men in health departments, city engineering and public works departments, and in utility operation, particularly water, sewage and refuse treatment and disposal plants.

Unfortunately time was lacking for any constructive consideration of these problems at that time. A draft of a formal report of the Committee was prepared for its consideration at its Cincinnati meeting. This was reviewed and discussed in detail. A new and revised draft was prepared to embody the ideas inspired by that meeting. This draft was submitted for criticism and comment to the members of the Committee, the members of the Executive Committee of the Sanitary Engineering Division, and to ten outstanding members of the Engineering Section, American Public Health Association and of the Conference of State Sanitary Engineers. In this form it was submitted, as a Progress Report, to the Sanitary Engineering Division at its session on January 16, 1941, held in connection with the Society's Annual Meeting in New York.

The present report has been revised to embrace the suggestions made in response to the request for comments and criticisms.

Nature and Scope of This Report

The Committee considers the present report to be in the nature of a "completion" report, of wider scope, more formal, and more comprehensive than

[3] *Civil Engineering*, April, 1935, p. 229.

any which has been attempted in the past. In general, it purports to outline the problems and activities of the sanitary engineer; to discuss the matter of his education and training; to indicate the present extent of opportunities for his professional activity and public service; to appraise and forecast the future possibilities of those services and endeavors; and finally to recommend means of improvement of the status and field of opportunity of the sanitary engineer.

General Survey of Problems and Activities of the Sanitary Engineer

The partly developed and the potential activities of sanitary engineers may perhaps be named and classified in ten categories, as follows:

(1) The quantitative and qualitative control of the air supply;

(2) The quantitative, sanitary, and esthetic control of the water supply;

(3) The control of the milk supply—production, transportation, pasteurization, and handling;

(4) The control of other food supplies—production, transportation, preservation, and handling of food products;

(5) The control of liquid wastes—sewage and industrial wastes; sewers, drains and treatment plants, ultimate innocuous disposal;

(6) The control of solid wastes—collection, transportation, treatment and disposal of municipal refuse, including garbage, rubbish, ashes, street sweepings, and dead animals; sanitary disposal of excreta;

(7) The control of the animal and insect carriers of infection—rodents, dogs, goats, cattle, hogs, etc.; preventive and palliative measures against flies, mosquitos, fleas, lice, ticks, etc.;

(8) The provision of environmental cleanliness—street paving and cleaning; dust, soot, and smoke control; swimming pool and bathing place sanitation; camps and camp grounds;

(9) The provision of sanitary conditions in factories, shops, schools, churches, theaters, and houses; rural sanitation; and

(10) The control of nuisances and other unsatisfactory conditions including odors, obnoxious gases, excessive noise, etc.

Virtually all of the activities comprehended in certain of these categories are almost exclusively of an engineering nature and, for their most effective prosecution, demand the services of the trained sanitary engineer. In other categories, the services of the sanitary engineer are still vital but they must be pursued in collaboration with other trained personnel such as biologists (bacteriologists, entomologists, and others), chemists, epidemiologists, health officers, police officials, and veterinarians. In still others the work of the sanitary engineer is perhaps less conspicuous although important in certain phases. In other phases the engineering method of approach is valuable and the procedures utilized should be of the same order in respect to rationality and regimentation.

All of these activities deal with the control of man's environment. All are therefore important. Some exercise a profound effect upon health and reduce the sickness and death rate from certain specific diseases. Others are con-

cerned more particularly with environmental esthetics. They have to do with the pleasure, satisfaction, and comfort of living and have little direct or immediate effect upon personal or public health. William T. Sedgwick's classical dictum[4] that public hygiene is concerned not merely with the prevention of premature death but, as well, with the promotion of normal life, health, and happiness, testifies to the significance of all of these undertakings whether they are to be classed in the category of disease prevention or whether they are to be defined as in the realm of esthetics.

Definition of the Term "Sanitary Engineer"

It is the judgment of this Committee that the sanitary engineer must be in very truth an engineer. As a sanitary engineer he must be competent to perform effectively at least some of the more important of the aforementioned undertakings in the various categories of problems and activities of sanitary engineering, comprehensively considered.

The sanitary engineer, moreover, according to his age and experience, should be eligible for one of the three regular grades of membership in the Society, namely, Junior, Associate Member, and Member. In this connection it should be understood that corporate membership in the Society is open to civil, military, naval, mining, mechanical, electrical, or other professional engineers, to architects, and to Marine architects. The only requirements for these relate to character, education, and experience. It is permissible that proper experience be substituted for education. Thus the Society has on its membership rolls many men who were primarily trained as chemists and biologists, or in other technical fields, but whose interests and experience have been such as to make them eligible to Society membership.

Still further, the sanitary engineer, after having attained the prescribed age and experience, must be capable of passing the tests for registration as a professional engineer in any of the states requiring registration of engineers.

The Title Sanitary Engineer Versus Public Health Engineer

In recent years certain groups of engineers have elected to call themselves "public health engineers" rather than to retain the older and time-honored title of "sanitary engineer." The evident purpose in this change of name has been to designate their interests as in certain of the more recently developed fields of environmental sanitation such as industrial hygiene, the control of milk and other food supplies, and the control of insect and animal carriers of infection. For example, the Engineering Section of the American Public Health Association, as currently entitled, was termed for a period of years (ending 1939) the Public Health Engineering Section. Its members were then presumably so-called Public Health Engineers. A majority of these are probably also members of the Sanitary Engineering Division of the Society and are there known as sanitary engineers.

The terms sanitary and sanitation are etymologically all-inclusive and there would therefore seem to be no linguistic reason why the designation sanitary

[4] "Principles of Sanitary Science and the Public Health," by William T. Sedgwick, New York, N. Y., 1903, p. 18.

engineer should not imply one who is interested in every phase of the control of the human environment in so far as the engineer and engineering are concerned therewith.

This Committee believes that all engineers so concerned should adopt a single designation for themselves, if for no other reason than to avoid confusion in the minds of the public and to bespeak a professional solidarity. It is believed, further, that this matter is of sufficient importance to warrant full and frank discussion with the hope that one designation only can be chosen and undeviatingly accepted as a professional title.

The designation "public health engineer" was probably adopted with the intent to imply training and capacity in the broader aspects of sanitation as well as in the fundamentals and practices of engineering. Moreover, it is possible that the term has a greater appeal to the medical profession than has the term "sanitary engineer." By precedent it is probably true that the title sanitary engineer has not fully represented the modern broad concept of his fields of activity and service. However these things may be, it is highly important that, whatever efforts are made in the direction of standardizing the title "sanitary engineer" as all-inclusive, they must be undertaken at once because the term "public health engineer" has already been given a certain degree of legal status in civil service and public health laws and ordinances.

The Academic Education of the Sanitary Engineer

General Statement.—The formation of this Committee developed from the fact that engineers were observed to be conspicuously absent from groups whose advice is sought in the formulation of far-reaching programs of public interest, involving much that is engineering, such as the conferences that led to the drafting of Title VI of the Social Security Act.

It may, alas, be true that one cogent reason for the absence of engineers from such conferences is that, to the present time, the impact of their personality and work, as a group, has not been sufficiently impressive. Too many engineers either have not been conspicuously capable, at least in the fields under discussion, or they have not made their work so significantly effective and important as to force recognition of capability on the part of the public. One of the most plainly marked avenues of advancement of sanitary engineering, then, is one which leads to the improvement of the quality and standards of its practicing personnel.

Therefore, the discussion of the academic education of the sanitary engineer will be pursued with a view to consider those aspects that would seem to have the greatest bearing on the development of leadership among sanitary engineers in fields of public interest wherein the work of the engineer is involved.

When dissatisfaction is felt with accomplishments in a field of endeavor, it is not uncommon to dispose of the matter with the broad allegation that the trouble lies in the training of the personnel. Yet in view of the frequency with which such allegations are made, it is amazing to find how little of constructive suggestion is offered.

Training should be considered from the standpoint of essentials which are basic and which will contribute to the development of sanitary engineers

equipped to earn for the profession a more adequate recognition as an entity in the public life.

Although it is recognized that those who were responsible for the early developments in sanitary engineering were not themselves trained as sanitary engineers, but that sanitary engineering training has been developed on the basis of works that they conceived, it is a fact that certain things can be clearly identified as specific, whereas others are related to preparation for sanitary engineering service.

In a paper presented in 1941[5] Arthur B. Morrill, M. Am. Soc. C. E., opens with the following terse statements:

> "This paper is based on two simple ideas, as follows: (1) that sanitary engineering is a distinct branch of the engineering profession, not a modification of civil engineering, and has suffered from failure to observe this distinction; (2) that the number of students of basic sanitary engineering subjects in undergraduate courses is too small, so that vocational guidance efforts are necessary to correct the situation."

Whatever the training is, it is obvious that it should be of such quality and content that it will develop leadership in so far as this characteristic can be fostered by education.

The training and education of the sanitary engineer must involve:

(1) Basic engineering, with perhaps less emphasis on such topics as surveying and more emphasis on such subjects as economics, statistical and computation methods, business law, and contract procedure;

(2) Sanitary engineering, proper, with its bases of bacteriology, microscopy, of water and sewage, physical and sanitary chemistry, etc.; and

(3) Environmental sanitation, with its implied basic courses in entomology, epidemiology, sanitary science, vital statistics, etc.

The academic training of the sanitary engineer will be discussed under the captions "Basic Technical Equipment," "Human Significance of Engineering," and "Expression."

Basic Technical Equipment.—A survey[6] of 123 engineering colleges in the United States shows that the average percentage allotment of curricula time for twenty-five colleges is as follows:

Subject	Percentage of total time
Cultural	9.5
Pure science	28.3
General engineering	44.5
Sanitary engineering	10.3
Public health	1.2
Miscellaneous	6.1
	100.0

[5] "Vocational Guidance and Sanitary Engineering Instruction," by Arthur B. Morrill, presented to the Sanitary Eng. Section of the Society for the Promotion of Engineering Education, Ann Arbor, Mich., June 24, 1941.

[6] "Survey of Undergraduate Engineering Training in Public Health and Related Activities in Engineering Colleges of the United States," by A. P. Miller, *Reprint No. 2021*, U. S. P. H. S.

The report states:

> "It is interesting to note that the Civil Engineering courses of 45 additional colleges include sanitary engineering subjects which require from 2.5 to 8.4 percent of the students' time, and that similar curricula in 19 other colleges demand that the student devote 2.5 percent or less of his time to subjects of primary importance to sanitary engineers. No two courses are exactly alike, indicating that our educational institutions follow no stereotyped schedule."

Although it is perhaps best for the present that the curricula in the various institutions should be not too rigidly stereotyped, for the reason that the best program of studies may not yet have been discovered, it would seem that this Committee, or some other body such as the Sanitary Engineering Section of the Society for the Promotion of Engineering Education, should set up some minimum standards of time and effort which should be applied to the various groups of subjects listed previously as undergraduate courses.

Reviewing the curricula in some institutions, it is a startling and pitiful fact that so-called sanitary engineering degrees are being conferred upon virtual civil engineering graduates with only the slightest exposure to those subjects which must be deemed essentially basic to sanitary engineering. In other words, it may be said that, although it is indeed shocking to discover what subjects are taught in some institutions to produce sanitary engineers, it is still more disconcerting to discover what subjects are omitted altogether.

Considering pure science as preparation for general engineering subjects, it is significant to note that an average of nearly three fourths of the time is allotted to general engineering subjects in the twenty-five colleges showing the greatest amount of time devoted to sanitary engineering. Since the sanitary engineer should be first of all an engineer it must be concluded that any curriculum devised for his proper training should be one in which a very considerable proportion of the time should be devoted to these general engineering subjects, but quite possibly not as much as that indicated above, and quite possibly, also, with a shift from those courses of less direct value to some which have more tangible merit.

The first point of departure of the sanitary engineer from the civil engineer with respect to structures is that he is concerned with the functions, the design of a structure itself being incidental to the processes to be controlled within it. Since these processes frequently involve the adjustment of chemical reactions or the behavior of bacteria under controlled conditions, it is essential that the training of the sanitary engineer include courses in sanitary bacteriology and sanitary chemistry, with facilities for the demonstration of their application in the processes concerned.

The purification of water and the treatment of sewage and industrial wastes have occupied such an important place in sanitary engineering that other phases of sanitation, requiring engineering treatment, have not generally been included in sanitary engineering curricula. Unquestionably this has been a mistake which should be promptly rectified everywhere.

A reasonably comprehensive curriculum should include instruction in the engineering aspects, the principles of design, and the operation of equipment for the pasteurization of milk, the control of mosquitos, rat-proofing buildings,

lighting and ventilation as concerns health and comfort in the factory and elsewhere, the sanitary principles of plumbing, garbage and refuse disposal, smoke abatement, and street cleaning. Altogether too often these problems are handled in practice without the benefit of sound engineering study, design, and supervision because there are few engineers who have had training designed to meet this need.

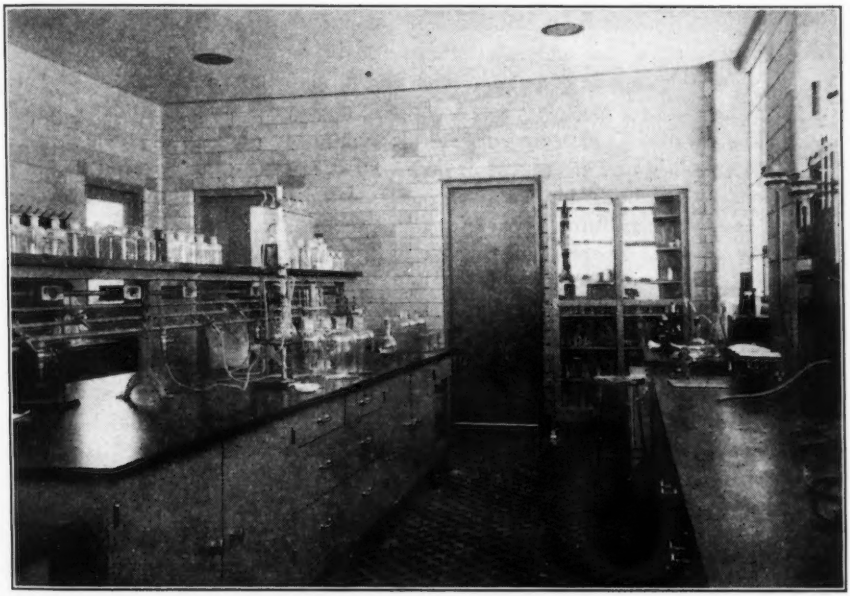


FIG. 1.—IT IS ESSENTIAL THAT THE TRAINING OF THE SANITARY ENGINEER INCLUDE COURSES IN BACTERIOLOGY AND CHEMISTRY, WITH FACILITIES FOR THE DEMONSTRATION OF THEIR APPLICATION (SEWAGE DISPOSAL LABORATORY, DENVER, COLO.)

At present it would ordinarily require courses in an agricultural school, a department of mechanical engineering, and a department of electrical engineering, to cover only a part of the subjects mentioned in the preceding paragraph. Furthermore, the student would probably have to take more than one course in heating and ventilation, for example, only portions of which would be suited to his needs. Obviously this is impractical, and the proper answer is a course or courses with suitable content bearing specifically on the particular phases of these subjects that are of sanitary engineering concern.

Human Significance of Engineering.—All sanitary engineering works are designed to contribute to the comfort or convenience of man or to safeguard him from disease. Therefore, it seems that courses dealing with man as an individual and in groups would help the sanitary engineer to carry his thinking on through to the relationships that the structures he designs and the processes he perfects have with the maintenance and development of the social structure. In short, it is hoped that he may emerge from the status of "scientific mechanic," simply designing works planned by some lay individual or group, to a position

of leadership wherever the possibility of a contribution by sanitary engineering to society exists. To this end courses in the following fields, at least, are indicated: Public health, sociology and economics, and political science.

As to public health, the sanitary engineer should have at least a basic understanding of epidemiology and vital statistics in order to understand adequately the reasons for and the potential health protective value of the works with which he is concerned.

The Sub-Committee on Educational Qualifications of the American Public Health Association has suggested these and other subjects in public health, but this committee was concerned with the preparation of engineers for work in public health as a specialty, and has therefore indicated more study in public health than would be expected of sanitary engineers in general.

As to sociology and economics, if the engineer is to visualize his undertakings as a part of, and build his works into, the social and economic structure, he should have some training in sociology and economics.

As to political science, since most of the work with which the sanitary engineer is concerned depends in some way upon the scope and limitation of the powers, and upon the processes by which governmental agencies function, it is believed that he should have some instruction in this field.

Expression.—The average engineering student, if left to his own devices, develops "transit and slide-rule-itis." He will rarely take more than the prescribed courses in English, journalism, and public speaking. Yet adequate training in these subjects frequently provides the element that permits men so equipped to become leaders as compared to others equally equipped scientifically who remain professional "technicians or mechanics" because of the lack of adequate ability in expression. Therefore, it is believed that more general attention to training in English, journalism, and public speaking would aid materially in equipping the sanitary engineer to exert a more potent influence and therefore enhance the recognition of his service wherever the public interest is concerned.

Conclusions Relative to Academic Education.—Holding in review the preceding analysis of conditions and problems relating to the academic education of the sanitary engineer, it would appear that the following general conclusions are tenable:

(1) The proportion of time devoted to general engineering studies in the best of American curricula may be slightly excessive; but it might not be unreasonable if less immediately valuable subjects now found in such curricula could be abandoned in favor of some that could be classed in the same general category but which would have greater educational merit under present-day conditions and pressures. Whether this proportion can reasonably be diminished in absolute terms, to make place for desirable specialization and broadening subjects, may well constitute a matter for further investigation.

(2) Subjects selected to give the engineer basic technical training to deal with the engineering phases of other problems of environment, comparable with that now provided with respect to water supply and sewerage, are to be recommended.

(3) Subjects designed to establish a better comprehension of the relationship of sanitary engineering to the major public interests, such as epidemiology, vital statistics, political science, sociology, and economics, are to be recommended.

(4) Studies designed to develop capacity for expression and leadership, such as English, journalism, and public speaking, are to be recommended.

(5) A survey of the present status of sanitary engineering training reveals such a lack of uniformity of curricula as to indicate the impracticability of attempting to suggest a specific schedule that would be generally applicable. Furthermore, it is believed that most desirable progress may be expected through the individuality of approach by the different institutions, provided that a certain minimum amount of time is devoted to the basically essential courses mentioned hereinbefore.

(6) Finally, it is obvious that, even with a reduction in the proportion of the time now devoted to general engineering study, there is not time available in a four-year college course to treat adequately all of the subjects indicated as desirable.

This naturally raises the question as to whether four years is sufficient time in which to supply the student with enough equipment to enable him to command the place that this Committee seeks for him in the increasingly complicated affairs of these times.

If the additional interests outlined previously may be considered sufficiently important to be included in the curricula of institutions proposing to offer adequate courses of training for sanitary engineers, the problem becomes one of determining how they can be so included. If it should be found that the essential items cannot be comprehended by a four-year undergraduate college course without undue sacrifices of other basic requirements, the problem will require one of two solutions: (*a*) Provide extra time by extending the undergraduate college course beyond the four-year period; or (*b*) limit training in the undergraduate course of four years to basic general technical engineering and cultural courses and add one or more years of postgraduate study as necessary for sanitary engineering training. Most of the specialization study, and the sanitary engineering option courses now included in undergraduate study, in such case would be deferred until the graduate period.

It is not proposed to conclude that either of these alternatives is now necessary. However, it is suggested that if the four-year period is not adequate, consideration should be given to extension of the training period. In this connection it may be stated that the competition among institutions of higher learning, so-called, for students in sanitary engineering is so great that no one institution alone can organize a five-year or six-year course and expect to attract students. It seems to have resolved itself into a case where uniformity of procedure is essential.

In some institutions it is becoming increasingly common for students in sanitary engineering to extend their courses to four and one-half or five years or even longer. This additional time is usually elected to reduce the rate of subject-taking; that is, the number of units of work per unit of time. The

aggregate of subjects taken is commonly thus increased and desirable electives in the chosen field are thus permitted.

Actual experience in some instances has indicated that the most valuable results of postgraduate study have accrued to those who have had some practical field experience (say, three or four years) before undertaking such work. These men are likely to apply themselves more diligently and to recognize their educational deficiencies more poignantly.

Early Graduate Experience and Practice in the Field

For those who have pursued competent undergraduate curricula in sanitary engineering the best general postgraduate training will commonly be found in professional practice. However, such experience should be obtained under the close supervision of able and interested engineers whose own training and experience have been of the proper kind and whose standards are of a high order.

Exceptions to this dictum might be represented by sanitary engineers who enter health departments in which competent engineering supervision will be unavailable.

Other exceptions may be represented by sanitary engineers who propose to enter more or less highly specialized fields at the outset of their careers. Among these fields are teaching and some phases of health department work such as industrial hygiene, the control of milk and other food supplies, the control of animal and insect carriers of infection, and the like. For those persons, graduate work in sanitary engineering, leading perhaps to a higher degree than the bachelor degree, may be highly desirable. Moreover, it is extremely important that instructors in the engineering phases or aspects of sanitary engineering acquire practical experience in the design, construction, and operation of sanitary works early in their careers.

Sanitary Engineers in Federal Health Service

The U. S. Public Health Service (of which the National Institute of Health with its strong research facilities has become a very important part), has fostered the growth of sanitary engineering activities within the Service and in the states during several decades until there is now an influential, loosely interlocking, functioning "organization." This "organization" permeates and influences sanitary engineering progress, through the state services, in district, municipal, and local political subdivisions by example, financial aid, participation in programs, advice on and review of policies, and by technical help.

The status of the sanitary engineer in the Service has been steadily improved through this period with the expected result that men of ability have been attracted who are able to exert leadership in their dealings with state and local officials. Often, through interchange, the good points of programs to improve state sanitation have been taken from a state division showing initiative worthy of emulation and introduced into other states. A steady influence upon states to further programs more adequate in scope and application has been exerted. Due in no small measure to efforts of the Service there are now sanitary engineering activities in all forty-eight state departments of health. This was far from true a very few years ago.

The sanitary engineers of the Public Health Service have regulatory supervision over water supplies and foods on interstate carriers, advisory supervision over shellfish, and they provide sanitary engineering services to other federal agencies. Some of the activities covered (direct, not through states' relations) are water supplies on vessels, rat-proofing of docks, sewage disposal at national parks, and transportation of mosquitos in airplanes. Activities in states' relations cover the entire field of sanitary engineering, including industrial and community hygiene. Outstanding federal health engineering contributions, in both research and application, to the needs of the field have included milk supply sanitation, stream pollution, and industrial sanitation.

The personnel of the Service includes approximately ninety sanitary engineers, of which number slightly more than one fourth hold commissions in the Regular Corps of the Service. The remainder hold either Reserve commissions or have Civil Service status. These engineers are engaged primarily in water supply, sewage disposal, shellfish and bathing place sanitation, recreational and rural sanitation, drainage and mosquito control, air hygiene, housing, sanitation of food dispensing establishments, and other aspects of environmental sanitation. The personnel in these fields includes many specialists in the various branches of the entire range of engineering as applied to public and industrial sanitation. This personnel is assigned to the several divisions of the Service. If there is any suggestion that this Committee might venture to make regarding the Service, it is that consideration be given to the strengthening of coordination of all sanitary engineering activities within it to the end that a more unified leadership might be provided for the entire profession. Increasing the number of officers commissioned in the Regular Corps would materially aid in strengthening the Engineering Corps of the Service, since it would place the engineering activities on a career basis.

As of January 1, 1941, there were only twenty-four commissioned engineers on active duty in the Public Health Service. Of these, fifteen were in the senior grade, four in the sanitary engineer grade, and five in the passed assistant sanitary engineer grade. It is significant that there were none in the assistant sanitary engineer grade. Of the fifteen in the senior grade, seven were between the ages 55 and 62. The retiring age is 64. The question might well be asked why young men are not being appointed to the lowest grade, eventually to take the places of those who will be retired. This situation does not obtain in the medical branch of the Service. It patently should not exist with reference to the sanitary engineers. Moreover, the criticism may justly be made that the total number of commissioned engineer officers is ridiculously small in view of the need for and value of their services. The commissioned engineering personnel should be given an opportunity for expression in all matters pertaining to environmental sanitation. This does not appear to be the case at present.

The activities of the Public Works Administration, the Farm Security Administration, and the several federal housing agencies and other federal projects, agencies, and enterprises have provided opportunities for considerable numbers of sanitary engineers, performing definitely as such. Statistical information is as yet lacking to this Committee upon this matter. The avail-

able evidence indicates that these represent important and rapidly growing fields of sanitary engineering service.

Sanitary Engineers in State Departments of Public Health

The duties generally assigned to the bureau or division of sanitary engineering in a state health department include the public health aspects of all types of environmental conditions whose control is based upon engineering principles regardless of the magnitude or technical difficulty of the individual problems involved. These duties have been classified and itemized in general terms hereinbefore under the caption "General Survey of Problems and Activities of the Sanitary Engineer."

In some states, the items covering milk and foods are under a separate bureau in the state department of health, or under one of the other state departments, such as agriculture.

The principal functions of the sanitary engineering personnel in state health departments can perhaps best be shown in a general way by the following classification:[7]

1. Administration.
   (*a*) Executive—Management of office, direction of personnel, budgets, purchases, etc.
   (*b*) Professional—Analysis of problems; planning of work; approval of standards and procedures; appraisal of qualifications and performance of personnel; professional guidance of personnel; development of codes, regulations, and other legislation, etc.
2. Supervision of environmental conditions (implies mandatory regulation).
   (*a*) General—Throughout an area or in general terms.
   (*b*) Specific—Regarding particular problems and involving specific details, etc.
3. Consultation (implies conferences, advice, and stimulation of voluntary action).
   (*a*) General—Group approach regarding broad problems, interpretations, etc.
   (*b*) Specific—Regarding individual problems and specific details, etc.
4. Promotion (implies specific "sales objectives," not mere educational efforts).
   (*a*) General—Group approach through publicity and meetings; promotion of legislation or appropriations, etc.
   (*b*) Specific—Individual projects or items of improvement; compliance with specific requirements, etc.
5. Education (includes popular health instruction and field instruction of public health personnel).
   (*a*) General—Assisting in health education program; preparation of material; field instruction of groups of student public health personnel, etc.

[7] See "Functions of Public Health Engineering Personnel, Progress Report of the Committee on Coordination of Public Health Engineering Activities," *Year Book*, A. P. H. A., 1939–40, p. 52.

(*b*) Specific—Individual approach in training workman; interpreting plans or explaining legislation; training student personnel in particular techniques, etc.

6. Investigations (includes technical studies, special investigations and sanitary inspections).
   (*a*) Fundamental research—Cooperative research projects regarding unknown factors or influences, etc.
   (*b*) Practical research—Study of known or accepted data to determine applicability in particular situations, etc.
   (*c*) Surveys—To determine details of sanitation service required or afforded in particular jurisdictions; to determine status of sanitation in given areas, industries, institutions, etc.
   (*d*) Routine—Sanitary engineering investigations; sanitary inspections, etc.
7. Design (includes public health features and sometimes all details of structures, machines and devices which are of importance to health).
   (*a*) Original standard plans for privies, dairies, screens, small engineering projects, etc.
   (*b*) Review and approval of water works, sewerage systems, plumbing, ventilation systems, buildings, impounded waters, milk plants, food plants, swimming pools, etc.
8. Office (non-administrative): (*a*) Clerical. (*b*) Drafting. (*c*) Miscellaneous.

All 48 states have sanitary engineering bureaus or divisions. The engineering personnel employed in these divisions ranges from one to forty one. The salary for the chief of the bureau ranges from $2,400 to $7,200 per annum. The salary for the assistant engineers ranges from $1,200 to $6,000 per annum.

It is very apparent that few state health departments, if any, have employed sufficient engineering personnel to do the work assigned to the engineering bureau or division. It is believed that, with complex problems resulting from the increasing density of population and with the recognition of the public health aspects of environmental conditions, the sanitary engineering personnel in the state health departments will generally need to be increased.

Sanitary Engineers in County and District Health Organizations

The duties and functions of sanitary engineering personnel in district and county health departments follow those of the sanitary engineering personnel in state health departments but, of course, with certain restrictions. In all probability a broad general policy for county and district health departments will be formulated by the state health department which will, however, unquestionably reserve the right of approval or disapproval of plans and specifications for certain engineering works, such as public water supplies, sewerage systems, milk pasteurization plants, swimming pools, etc., and for even small projects such as sanitary privies, cisterns, septic tanks, and the like, as well as of chemicals used for sanitary purposes, and of equipment; for example, that which is to be used in food manufacture or handling. It can readily be seen that this is absolutely essential, for it is obvious that it would not be to the

best interest of the public if each county or district health department in a state were permitted to use its particular ideas of design or of the efficiency and value of some product or equipment. The same holds true for the various types of inspection forms, such as those used for food handling places, public buildings, plumbing, milk handling plants, etc., for here, too, uniformity of inspection is needed for proper evaluation of the work.

In the United States there are 3,071 counties, of which number, according to the 1940 census, about 450 had populations in excess of approximately 50,000. Of these some 380–390 had populations exceeding approximately 50,000, exclusive of cities of about that same population.

It appears that there are more than 1,000 organized full-time county, unit, or district health departments, many of which should enlarge their effectiveness by establishing a bureau of sanitary engineering and employing trained personnel. Unfortunately only a small percentage of these employ full-time sanitary engineers. The opinion expressed by about two thirds of the state sanitary engineers seems to indicate that every county, county unit, or district, having a population of 50,000 or more, should have the benefit of full-time services of one or more sanitary engineers. In some cases, where problems of sanitation are for some reason acute or where the economic status of the population is high, sanitary engineers should be employed even though the population is smaller than the figure just named.

The figures presented in the preceding paragraphs would seem to indicate that there is a potential demand for a large number of well-trained and experienced sanitary engineers for district and county health departments.

The provisions of the Social Security Act, recently effective, have stimulated the demand for trained sanitary engineers in county and district health organizations. It has become necessary in some cases to send men to educational institutions for training in special branches of sanitary engineering in order to provide for the expansion of activities.

Sanitary Engineers in City Health Departments

The duties and functions of the sanitary engineering personnel in city health departments will generally follow those formulated for the sanitary engineering personnel in state, county, and district health departments, but of course with certain restrictions and also in all probability with more detailed effort being devoted to certain of those duties.

In the field of water supply, the city health department has the responsibility for the safety of the public water supply even though in many instances the municipal water supply is under another department of the city government or is privately owned. It is naturally the concern of the sanitary engineer to check the conditions relating to a water supply in order to avoid the possibility of any outbreak of water-borne diseases. He is, of course, extremely interested in seeing that the water after leaving the city mains does not run the risk of being polluted through cross connections in the plumbing. He should also regulate the many small subsidiary water supplies, such as private wells, springs, etc., which are used for domestic purposes, food establishments, and the like.

What has been said about water supply applies equally to the actual collection, treatment, and final disposal of sewage, refuse, and other wastes which, in all probability, will be under other city departments. It is the city sanitary engineer's duty to see that the necessary health requirements are fully met in executing these city functions.

In the field of milk and food protection, the absolute need of proper control of food manufacturing, handling, and dispensing is becoming well recognized and, in general, should be placed under the sanitary engineer. This work should cover not only routine inspections but should include the approval of new buildings, equipment, and methods of manufacture.

The demands in the field of industrial hygiene and sanitation upon the city health department are becoming ever more pressing. The control of dust, fumes, excessive temperatures, harmful chemicals, and the like, in shops and factories, as well as of the pollution of the general atmosphere in most large cities becomes chiefly the problem of the engineer, for in order to solve the problems presented, satisfactorily, it is necessary to have engineering guidance.

In any densely populated area, insect and rodent control becomes an important problem and effective measures should be formulated and supervised by the sanitary engineer.

A city health department should be consulted in city or regional planning problems, in the starting of any housing developments, or when the detailed designs of transportation systems, public utilities, drainage systems, etc., are being prepared. Here the health officer should have engineering advice.

No city health department can be considered functionally complete, or successfully operable, unless the foregoing activities are directed by a qualified sanitary engineer.

It seems to be the consensus of opinion that each of the 220 cities, which, according to the 1940 census, had a population of approximately 50,000 or more, should have the benefit of the full-time services of one or more sanitary engineers in an organized, properly equipped bureau. A survey made recently indicates that only 53 city health departments have sanitary engineering bureaus with but 66 engineers employed. In other words, only one fourth of the cities with a population greater than 50,000 employ sanitary engineers. This would indicate that there is a potential demand for a considerable number of sanitary engineers who are well trained and who also possess the necessary experience and qualifications.

In contradistinction to the apparent attitude toward the sanitary engineering group in the U. S. Public Health Service, indicated hereinbefore, and apropos of the immediately foregoing discussion, it is refreshing to note that in a report[8] on the public health of the City of Los Angeles, Calif., the U. S. Public Health Service recommends that the health department be organized into three main divisions: One for preventable disease, one for sanitation, and one for health centers. The proposed assistant director would have charge of health centers and the other divisions would be placed under two equal sub-directors.

[8] "Report of a Survey of the City Health Department of Los Angeles, California," by the U. S. P. H. S., April–August, 1939, pp. 415–416.

Sanitary Engineers in Industrial Sanitation

Air and industrial sanitation comprise a group of related engineering practices in environmental sanitation which have quite recently grown important. Industrial hygiene is a specialty jointly administered by the medical and engineering professions in which the hygienic problems of the worker are adapted to his sanitary environment and vice versa.

Air sanitation includes prevention of pollution of the air of communities by smoke, dust, fumes, odors, etc., and control of the quality of air in public buildings and industries. In industrial establishments in general, air sanitation is a major phase of the "industrial hygiene" program and requires, as does community air sanitation, the application of the principles of sanitary engineering, mechanical engineering, chemistry and physics, and often of bacteriology and physiology. Very much the same integration of talents and application of analytical data are involved as in the case of the design and operation of a modern mechanized sewage treatment plant, principles of sedimentation, coagulation, filtration and fluid mechanics being preeminent in both fields more often than not.

Industrial sanitation likewise demands the services of the sanitary engineer in the solution of problems of water supply, sewage and wastes disposal, cross connections and other special hazards of plumbing, of illumination, extremes of temperature, compressed air, and noise, and of specialized problems incident to the manipulation of toxic substances or processes involving them. Governmental administration of industrial hygiene programs requires of the engineer surveys and reports on all environmental conditions and potential hazards which may in any manner affect the health of industrial populations.

Engineers engaged in air and industrial sanitation are recruited mostly from the fields of chemistry and mechanical engineering but many sanitary engineers devote part or all of their time to practice or supervision of this specialty. Chemists and mechanical engineers in this field, unless their practice in it is narrow, become, through necessity, sanitary engineers in fact, if not always in name. This is especially likely to be true in governmental positions or in industrial positions which happen to be medically supervised. This trend becomes increasingly pronounced as the art develops from emphasis on research and measurement of the problem to its engineering control. Just as in the early days of emergence of sanitary engineering as a profession distinct from civil engineering, the bulk of recruits were drawn from chemistry and civil engineering, so now is the bulk of recruits for this new branch drawn from chemistry and mechanical engineering. Never have enough sanitary engineers been trained specifically for sanitary engineering to provide more than a nucleus of an adequate body for the country's needs, and the inertia of educational institutions may be expected to allow this circumstance to carry over into the new specialties unless this inertia is persistently overcome in some manner.

To date, thirty-five industrial hygiene units are operating in thirty state and territorial departments of health, two state departments of labor, and three city departments of health. These units employ sixty-nine engineers and forty-three other non-medical technologists. The engineers receive $2,400 to

$5,400 per yr, the average being about $3,600. The industrial hygiene and sanitation program is directly supervised by divisions of sanitary engineering in seven states and two cities. The remainder are under immediate medical direction (in addition to the medical direction afforded by the heads of the departments under which the units operate) with one exception. Engineers of the highest ability must broaden beyond this specialty or exercise it elsewhere to achieve commensurate careers.

The U. S. Public Health Service employs five engineers and twenty-seven non-medical technologists in industrial hygiene. The engineers receive $2,000 to $7,200 per yr plus retirement benefits. Some are commissioned officers with ranks corresponding to the military. The U. S. Department of Labor and the U. S. Bureau of Mines also employ industrial hygiene technologists.

No record exists of the number of industrial hygiene engineers engaged in preventive work in industries, under compensation boards and commissions, as private consultants, as teachers, by manufacturers of industrial hygiene equipment, by smoke abatement commissions, and foundations such as the Air Hygiene Foundation, etc., but most of these groups are beginning to be represented in the American Industrial Hygiene Association formed in 1939. Its membership totals 259, of whom 101 are engineers and 40 are non-medical technologists (not engaged in governmental positions). Thus there exists an incomplete record of a total of 137 engineers and 52 non-medical technologists engaged in industrial hygiene and sanitation. The number of engineers, in particular, is still growing.

For the sake of progress on a sound footing there is great need at this juncture of integration for the practice of engineers in air and industrial sanitation with the established and successful practice of the older existing sanitary engineering divisions and offices. This, in turn, requires that much larger numbers of sanitary engineers be trained with a broader background in the fundamentals of sanitation, chemistry, physics and mechanics, with less emphasis, of necessity, on surveying, construction, and civil engineering specialties. In addition to work in industrial hygiene and sanitation, such men would find an outlet in design and operation of mechanized sewage treatment plants and incinerators, manufacture and sales of sanitary equipment in all branches of sanitation, as well as in all phases of sanitary engineering work in health departments and in consulting engineers' offices. In addition, they would be admirably fitted to invade such diverse fields as insurance engineering, safety engineering, and the many employments that require young engineers with a broad engineering training rather than those with a too-early specialization in any of the more "classical" branches.

It would be unfortunate indeed if these activities, which can be broadly classed in the field of sanitary engineering, should be divisive with respect to the profession. Rather than to permit competition to spring up as between different groups in the profession, the interests and activities of which are certain to overlap at many points, it will be far better that coordination be effected at the outset in the interests of efficiency, of ultimate gain to all programs, and of professional solidarity.

Sanitary Engineers in Health Service—General

It is encouraging to note the awakening appreciation of the value of the services of qualified sanitary engineers in the general field of public health. This is attested by the resolution relative to sanitary engineers in environmental sanitation adopted by the Governing Council of the American Public Health Association at its meeting in Detroit on October 10, 1940. It reads:

> "Whereas essential measures for the adjustment and control of the great variety of environmental conditions which affect human health constitute a large part of the program of Federal, state, and local health departments and, in the aggregate, involve large expenditures of public and private funds, and
>
> "Whereas the economical and effective control of any public health aspect of the environment on a community-wide basis presents important engineering problems of investigation, analysis, design, organization and supervision, therefore be it
>
> "Resolved that in state and local health departments provision should be made for responsible supervision of all environmental sanitation procedures and activities by properly qualified public health engineers, and be it further
>
> "Resolved that the scope of responsibility of engineering divisions of state health departments should be expanded, and their cooperative relationships with other divisions of state and local health service and with other agencies should be extended wherever necessary, in order to provide competent engineering guidance and to obtain greater unity and effectiveness in the program for sanitation of the environment in the several states."

Sanitary Engineers in Consulting and Other Private Practice

This field normally embraces the design of sanitary engineering works such as water supply systems, water treatment plants, sewerage systems, sewage treatment plants, and, occasionally, refuse disposal works; the supervision of their construction; and, perhaps to a more limited extent, the supervision of operation.

The volume of this work varies largely with construction activity and is affected by such conditions as good or hard times, construction for unemployment relief, the money market, and many other factors. Unfortunately, the average or norm of construction work is not determinative. For example, the pendulum swung to the extreme of construction activity in the late 1920's; it swung as far in the other direction in the five succeeding years. Public Works Administration (PWA) and Work Projects Administration (WPA) then came into existence with the result that many offices were "clogged" with work. What the future has in store cannot, of course, be predicted.

It is obvious that an engineer cannot maintain his office and his staff unless there is work to do. A lessening of work means a reduction in the income of the consultant. In such circumstances he must reduce his staff and perhaps reduce the size of his quarters. For the occasional job he may hire draftsmen by the day or hour. He may cooperate with local engineers in order to get field survey data at less cost.

Under these conditions the relatively small enterprise may seem attractive. Among such are small dams, swimming pools, correction of undesirable water supply conditions, improving distribution systems by pipe-line extension and elevated tanks, sewers, supervision of the operation of water and sewage treatment plants, and other items of a like nature. The engineer who has the ability to locate these jobs and obtain a sufficient number of them to ease the difficulties caused by the scarcity of larger and more important tasks is fortunate.

The young engineer attempting to enter this field cannot be assured of steady work to any greater degree than can the young construction engineer. Unless he is so fortunate as to enter the field at the beginning of a cycle of four or five good years, he will be compelled to gain experience by a succession of jobs ordinarily of unpleasantly short duration. However, the experience which he may thus gain may be excellent if his connections are with thoroughly competent engineers.

This circumstance of rotation of jobs is not, of course, a peculiarity of consulting work. It occurs in almost all similar lines of endeavor. Moreover, the first few years after graduation may well be spent upon a variety of tasks. In that way, and in that manner only, can broad experience, so valuable in later years, be gained.

Perhaps the most useful knowledge that a young engineer can acquire is an understanding of what public service means. He should learn to appreciate the requirements and viewpoints of those for whom he works. He should endeavor to serve efficiently, even though such service may not appeal to his immediate interests. It is obvious that he should attempt to perform competently to the end that when any task is completed he will have gained in reputation and have created enduring friendships. The fundamental importance of downright honesty should not require emphasis. No engineer can hope for true success unless he possesses both moral and financial integrity.

Remuneration, in the earlier years, will be only moderate. What it may become later will depend almost entirely upon the individual. An engineer with genuine ability to meet public officials and gain their confidence can always earn a good living if he is technically capable. In active construction periods his income may be large. During slack times he must receive his proper share of the available work in order to obtain a living income.

One of the eventual rewards of consulting practice is independence. Another is the commonly fine standing which the consultant enjoys in his community and among his engineering associates. The traditionally high standards that the great majority of engineers observe yield splendid rewards in terms of public esteem.

In the office of one of America's leading technical journals some 1,030 consulting engineers are recorded as practicing in the water and sewage works fields alone. Some of these men do comparatively little consulting work inasmuch as the list includes county and city engineers who are employed only occasionally as consultants in addition to their regular work. The number of assistants employed by this group of consulting engineers is not known. It undoubtedly fluctuates markedly with the volume of public and private work that is being done.

Sanitary Engineers in Public Works Departments of States, Counties, and Cities

No clear picture exists, in the great majority of cases, of the functions and duties of sanitary engineers employed in the public works departments of states, counties, and cities. These men are generally engaged in a wide variety of work, although their specialized training in sanitary engineering may be a requisite for the positions which they hold.

In the states most of the sanitary engineering functions are grouped under state departments of health, but not always. Some state public works departments maintain their own sanitary engineering personnel. However, it appears to be more commonly the practice to rely on engineers employed in the health departments or to call in consultants. Promotion is likely to be slow and the pay limited.

Aside from a relatively few counties—certainly only a fraction of the number that there should be—sanitary engineers are all too rarely engaged. Sanitary districts, mosquito abatement districts, and occasionally drainage districts employ a relatively small number of sanitary engineers.

In the cities the principal opportunities exist in water and sewerage works. The former is the most likely to be under the local department of public works. The latter is likely to be a separate department of the city. A recent survey of 490 water works revealed that the superintendents of 157 classed themselves as engineers, whereas 333 stated that they were not engineers.

Reviewing the general situation, it appears that opportunities in this field have been far too limited and the openings too few, except for those who have the adaptability and capacity to perform work of wider scope than is ordinarily considered in this connection. The continuity of service is likely to be satisfactory but the remuneration may be relatively low for the sanitary engineer who fails to broaden his activities.

Aside from the foregoing desultory information in respect to water works superintendents, reliable data are lacking as to the number of sanitary engineers employed in this general field. This may be due in part to the fact that many of these persons, while performing in greater or less extent as sanitary engineers, have other duties also. These may often entail greater responsibility and better financial rewards than does or would straight sanitary engineering.

The situation in Indiana, as of January, 1941, as explained by B. A. Poole, Assoc. M. Am. Soc. C. E., chief engineer, Bureau of Sanitary Engineering, State Board of Health, is of interest in this connection and may be broadly illustrative of the conditions in other states:

> "Indiana now has about 325 public water supplies and 100 sewage treatment plants. Of these 325 public water supplies some 50 depend upon surface waters. Iron removal and softening plants are also beginning to be installed in this state. There are surprisingly few sanitary engineers connected with Indiana water works. I think it not unreasonable to say that Indiana water works can support and should have from 125 to 150 sanitary engineers in their personnel organizations.
>
> "Strange as it may seem, the caliber of operating personnel in Indiana sewage plants is, on the whole, better than the caliber of the water works

personnel. I attribute this to the fact that our sewage plants are all relatively new and we have been able to impress the 'city fathers' with the value of the engineers in their responsible positions. In fact, most of the sewage plants of any size have at least one engineer generally acting as superintendent. It is true these men have not all had sanitary engineering training, primarily because we did not start turning out sanitary engineers soon enough in Indiana. We eventually expect to have about as many sewage plants as we have water plants, although this goal will certainly not be realized for a number of years. However, I believe we can reasonably assume that in ten to fifteen years from now, Indiana sewage plants should be utilizing at least 100 sanitary engineers plus some sanitary chemists."

Sanitary Engineers as Salesmen of Technical Equipment and Products

The scope of the work in this field varies with the policy and the product of the manufacturer. The work may embrace a wide variety of design or it may be little more than straight selling of a more or less standard product, which actually can be handled by a man without technical training. At one end of the scale may be a pump manufacturer whose salesman merely quotes a price on equipment already selected by the consulting engineer; at the other end may be the maker of sewage treatment equipment whose representative actually aids the engineer in working out design factors.

There are probably more openings in this field for the sanitary engineer than in any other. Experience qualifications vary with the manufacturers' policies; some hire only experienced men, largely recruited from state health departments and consulting engineers' offices, whereas others take men direct from colleges. In either case, financial returns are fairly good, probably comparing favorably or exceeding those gained in public service. Continuity of employment also compares favorably with that in almost any other phase of engineering. One characteristic of the work should be mentioned. Few men, after service in this field, find ready openings in other engineering lines. Once engaged in sales work, the engineer is likely to remain in it, although not necessarily with the same manufacturer.

As usual, there are few data available regarding numbers of men engaged in this field of work. An approximation might place the number around 1,500. With the increasing tendency toward use of more equipment in water and sewage treatment, for instance, there will always be openings for men who have the talent and the liking for selling.

Some engineers, of course, are engaged by manufacturers primarily for design and not for sales. However, these men must realize that they, too, are truly salesmen. To make an organization successful it must sell its products and the rewards will usually go to those who directly or indirectly aid most in this essential.

In the past there was a feeling that an engineer suffered a definite loss of standing when he engaged himself to sell equipment. It is good to note that this feeling has largely disappeared. An engineer can remain honest and ethical in selling as well as in any other line of engineering work.

Sanitary Engineers as Operators of Sanitary Works

Recent canvasses of the water and sewage works fields by *Engineering News-Record*[9] have revealed the fact that there are existent in the United States some 3,100 water-treatment plants and 4,500 sewage-treatment plants of various kinds and capacities. The stated number of water treatment plants does not include those in which chlorination alone is practised, nor those in which odor control, only, as by the use of activated carbon, is exercised.

The larger works of both kinds are likely to be supervised by sanitary engineers or sanitary chemists. Since the tendency in all such plants is toward a high degree of mechanization, the services of engineers are becoming more and more essential to satisfactory operation. If the plants are of sufficient size and complexity, sanitary chemists should also be employed on the operating staffs.

The great mass of plants, however, both water and sewage treatment, are of relatively small size. For the most part the operation of such works has been entrusted to untrained men, with consequent mediocre operating results in far too numerous instances. It would be a wise and helpful procedure if young sanitary engineering graduates were employed here. The experience which they would gain would represent a good starting point for the assumption of larger responsibilities elsewhere. The resulting fairly rapid turnover of operating personnel would not be a serious matter because trained men can learn to operate such plants effectively in a very brief time and the operational results would probably be very much better than they are under the existing practice. There are few works indeed which are so small that a proper salary cannot be paid to young sanitary engineers immediately or shortly after graduation.

Statistical information is lacking with respect to the number of sanitary engineers engaged in the operation of water and sewage treatment plants and other sanitary works of like nature. That this number is woefully smaller than the logic of the situation demands is obvious to those who are acquainted with the existing situation.

The potential opportunities are large if responsible authorities can be made to understand the advantages that will accrue if trained sanitary engineers are employed for such work rather than untrained and too often not too intelligent laborers.

The Social Security Act, the Field of Opportunity, and Appurtenant Problems

The field of opportunity for sanitary engineers has taken forward strides since the passage of the Social Security Act. The results of this act began to be felt when the appropriation for health work increased sanitary engineering facilities early in 1936. Concurrently with this act the increase in construction of sanitary works through PWA and WPA augmented the quantity of work for the professional designer and professional operator of these works. This increase strengthened the demands for allocation of social security funds, earmarked as health funds, for the specific field of sanitary engineering. The

[9] *Engineering News-Record*, September 28, 1939 (water), January 19, 1939 (sewage).

grants for sanitary works stimulated construction both with and without actual acceptance of federal funds.

What the net effect of the Social Security Act has been to date on sanitary engineering, distinct from concurrent stimuli, it is impossible to say. Also no attempt will be made to present a quantitative picture of the tremendous impetus which sanitary engineering has been given in these few years, especially in the public health field. From small beginnings the industrial hygiene branch of sanitary engineering services in health departments has grown tenfold during this short time, until three fourths of the states have full-time units (exclusive of the few states with too small an industrial population to warrant it at this time). Some sanitary engineers hold that the scope of the profession has doubled in the past several years.

Educational, research, administrative and applicational aspects have been greatly strengthened. Forty-eight states now have sanitary engineering personnel in their health departments. Municipal, district, and county units have begun to improve their environmental health conditions with sanitary engineering personnel. The level of technical ability of their sub-professional assistants has been raised. Health administrators have come to take for granted, for the first time on a nation-wide basis, that engineering personnel in local health departments is essential to successful results. It remains for the engineering profession to add its weight to the extension of these professional services into hundreds of communities.

Although the quantitative picture to date is unavailable this is not the prime desideratum. Future gains, or consolidations of past gains, should be the immediate concern of this Committee. Action should be based on the assumption that sooner or later the artificial stimuli to advancement of sanitary engineering will cease. At such time the profession will have to stand entirely on its own feet. Where the new services will have proved themselves they may be expected to survive; where not, to expire. Immediate action of the Committee should be directed toward the initiation of a movement for the improvement of these professional services.

Continuation of professional services depends upon two things: There must be a demand and the demand must be fulfilled. The profession cannot do much to create the demand beyond indirect advertising and demonstrating the existence of instrumentalities for satisfying the demand. If it had any unified direction, the profession could do much toward the fulfillment of the demand in an efficient manner so that future demands would increase rather than be inhibited at the outset. This requires that the professional services rendered be of a high order. To achieve professional services of a high order requires that the services be rendered by professional men of specialized training and ability. The rapid growth of these services has led to a far too widespread practice of this rigorous specialty by the semi-qualified. Much of the recent progress has been sound, and much unsound. It is specifically here that professional engineering direction of policies in this expansion can do the most good. It is here that the need is the most acute.

One of the first requirements is to differentiate between professional sanitary engineering and the attempt to practice this art by the many semi-qualified

engineers and near-engineers. The representation on a nation-wide scale that sanitary engineering is being practiced in a community in full view of all the community leaders will prove devastating to the future of the profession if the majority of these semi-qualified newcomers do not make good. The entry of the newly qualified man into the field will be made more difficult. The level of all practitioners will be pulled lower. That this contingency is not remote may be made evident by exhibition and examination of much that passes under the label of engineering in the public health field.

Standards have been outlined and given prominence by sanitary engineers for the reason that this danger was felt. Sub-professional practice below these standards is needed, but cannot safely be allowed to pose as that which it is not. There is no organization to compel compliance with such standards outside of the engineering profession. Sanitary engineers as a body have done remarkably little to remedy this serious evil. The engineering profession of which they are a part has had no program offered them which they might support. One of the most important functions of this Committee is to initiate such a program.

In the onrush of sanitary engineering to broader fields little pause for an inventory of accomplishment, coordination, and relation of services to problem has been made. In general, the progress has proceeded in "fits and starts." Where skilled services were available and could be had, these would be employed. Meanwhile, a gap would obtain where sanitary engineering had a real job to do, yet the gap would not become manifest. In few, if any, reports of sanitary engineering divisions of state departments of health is an adequate picture presented of the unsolved and unattacked engineering problems existing in the states outside of the specialties in which the division deals. In general, there has been no coordinated engineering study of the over-all services of the state, district, and local agencies; and where new services have been initiated there has been insufficient integration into preexisting sanitary organizations. Attempts have been made on the part of various organizations, representing groups of engineers concerned, to secure improvements in this regard, but while a little of accomplishment has resulted, these sporadic and scattered attempts have been rather futile. Here is another function which such a Committee as this should assume: namely, to give direction to these sporadic efforts to build within the framework of the public health organization a strong engineering force capable of cooperating with and assisting (in its proper sphere in a manner creditable to itself and therefore of enhanced value to the field of public health) the administering medical profession with which it commonly works.

Cooperation Among Sanitary Engineering Groups

The highest development of sanitary engineering as a branch of the engineering profession, and the most satisfactory status of its practitioners, can be consummated only through coordinated efforts: (1) Of the individuals themselves; (2) of their own technical organizations; and (3) of the entire engineering profession.

There are four organized groups of strictly sanitary engineers plus a certain proportion of members who have indicated an active interest in sanitary engineering. The largest of these is the Sanitary Engineering Division of the Society. At the present time the membership of this Division is about 1,250. It is the sixth in point of numbers among the twelve divisions into which the membership of the Society is segregated on the basis of announced primary technical or professional interest. The next largest group of engineers whose immediate interest is sanitary engineering is the Engineering Section (until recently called the Public Health Engineering Section) of the American Public Health Association. The current membership of this Section is approximately 700 (680 in April, 1941, the section membership constituting about 12% of the Fellows and 10% of the Members of the Association). The third group in question is the Conference of State Sanitary Engineers whose present membership is about 160. The fourth group is the recently organized Conference of Municipal Public Health Engineers.

In addition to these four groups composed almost entirely of sanitary engineers, large numbers of sanitary engineers are members of the American and the New England Water Works Associations and the various sewage works associations now amalgamated in the Federation of Sewage Works Associations patterned somewhat after the organization scheme of the American Water Works Association.

No cooperative effort as between these several groups looking toward the improvement of the status of the sanitary engineer, or of this branch of the profession as a technical enterprise, has ever been made. This has been due in part, perhaps, to apathy and inertia; but more particularly it has probably resulted from the fact that there has been no motivating body, such as this Committee might well be, to formulate a program and stimulate continued interest therein.

Publicizing the Sanitary Engineer

The activities and capabilities of trained and experienced sanitary engineers in the fields of water supply and water treatment, sewerage, sewage treatment and disposal, refuse collection and disposal, mosquito control, and other enterprises of fairly long standing and a currently high degree of development, are comparatively well known to those responsible for the employment of that type of service. It is in those features of environmental control which more recently have come to be recognized as having a significant bearing upon public health, broadly interpreted, that the education of employing authorities as to the value of the sanitary engineer's services is particularly demanded.

Obviously, sanitary engineers as a group cannot afford to permit any province of potential opportunity for service to lie fallow. The greatest present need, however, would seem to be to demonstrate the capability of the properly trained sanitary engineer in the engineering phases of industrial hygiene, air, building, camp and swimming pool sanitation, insect and rodent control, milk and other food control and sanitation, noise prevention, and the like.

The persons to be reached and educated with respect to such matters are health officers, the medical profession in general, public health personnel in

general other than engineers, and the engineering profession as a whole outside of the sanitary engineering group.

Publications such as that of the Institute of Research, Chicago, entitled "Sanitary Engineering As a Career," dated May, 1938, can be of paramount value if revised to include this larger program of the sanitary engineer's activities and interests and if given wide circulation among those who should be reached. This Committee can assist materially in the circulation of such educational material if it will prepare a mailing list of persons to whom it should be distributed.

On the other hand such publications as the brochure entitled "Engineering—A Career, A Culture," issued by Engineering Foundation and widely circulated, has perhaps done positive harm to sanitary engineers and sanitary engineering in that they and their activities are dismissed with a single paragraph under the general subject of civil engineering. The comment has been made that "the average high school student examining the pamphlet will reach the conclusion that he may successfully practice sanitary engineering after graduating from a typical civil engineering course." Is it possible that it is the fault of sanitary engineers themselves that the true nature of sanitary engineering is not better understood, even among fellow professional engineers in other branches?

The Need for Vocational Guidance

The recent demand for basically well-trained sanitary engineers in general has been far in excess of the supply. At the present moment the need for sanitary engineering officer personnel in the Sanitary Corps, U. S. Army, is acute. The total number of such men graduated from the educational institutions in the United States may, in truth, be said to have always been inadequate if the real needs of the country are considered rather than the apparent or superficial numbers of service openings. In other words, conditions would have been vastly better had this greater supply of well-trained sanitary engineers been available and utilized.

These facts point to the need for vocational guidance, as well as a better comprehension, on the part of the public and of employing agencies and persons, of the potential value of the services of such well-trained men.

In reference to this matter, Mr. Morrill[5] has made the following comments:

> "As for the need of vocational guidance, it seems obvious that the number of opportunities is increasing for men with sanitary engineering training. Studies show, however, that in increasing proportion these positions are being filled by men whose basic training was in some other line, usually civil engineering. Enrollment of graduate students in sanitary engineering is increasing, usually of men without the fundamental training in chemistry and biology needed to make their graduate work most effective. Too much of the time of these mature students is spent in making up deficiencies in those subjects which could have been better studied in their first or second college years. And at the end of five years of work the students have a training which, for their chosen lifework, is often no better than could have been obtained in four years if a proper start had been made.

"This unfortunate situation must be due to one of two causes. Perhaps the men in question did not know, as freshmen, enough about sanitary engineering to realize that there was where their true interest lay, rather than in civil engineering. Or perhaps they made the natural mistake, under present conditions, of thinking that sanitary engineering was simply an offshoot of civil engineering, to be acquired by specialization in the last two years, after a start in the regular curriculum of civil engineering.

"It should not be necessary to urge the need for vocational guidance. Most of you are probably only too familiar with the inadequate reasons that determine for many young engineering students their choice of course. Something becomes the fashion of the day and every waverer takes it, without much reference to his own taste and aptitude for the subject and still less for the demands for graduates in the field. It was a surprise to the writer to learn recently that at one good engineering school it was expected that the student entering would have made a tentative selection of his branch of engineering during the latter half of his last year in preparatory school. Unless high school students are much more mature and far-sighted than they used to be it seems certain that in many cases such a selection will be a poor one. In most cases it should be possible to delay the final selection till the end of the first college year, and even then the student should be given all possible assistance in arriving at a correct decision.

"In several cities programs of vocational guidance are under way, often under the direction of the local engineering society. There should be many more of these. That at Detroit is directed particularly to students in their last year of preparatory school. It is intended to assist them to decide whether they shall go to an engineering school,—only incidentally to decide what kind of engineering they shall study. In theory the program aims as much to discourage young men who would not make good engineers as to encourage those who are well suited to the profession. Because of the natural enthusiasm of the speakers and counsellors selected, it is somewhat difficult to prevent them from becoming salesmen for their particular brand of engineering.

"There is first a general talk by an engineer or engineering educator of national reputation, after which the students are invited to call at one or more of the classrooms in the building assigned to different branches of engineering. Advisers are appointed by the different cooperating engineering societies; mechanical, electrical, chemical, civil, etc., and a sign at each door indicates the kind of engineering being discussed within. The advisers are always practicing engineers, never teachers of engineering. Perhaps it is a matter of professional modesty on his part, but the engineering professor who has headed the work for some years maintains that this selection of practicing engineers makes a distinct impression on the students and increases their interest.

"Such vocational guidance programs, wherever they exist or can be promoted, are an excellent field for work to increase sound enrollment in undergraduate courses in sanitary engineering. It is perhaps significant

that, although the writer has acted once or twice as an adviser by appointment of the civil engineering society, it never occurred to him that he should rather have been explaining the distinct branch of sanitary engineering, with a room and door sign all its own. It should never be forgotten that the number of students needed is not large, although the importance of getting good material is great. It has been estimated that 200 well-qualified graduates per year in sanitary engineering would be quite sufficient for the positions available. This means, however, that in every large city a few preparatory school students should be considering sanitary engineering and that the field should be brought to the attention of promising young men in every city large enough to have a local engineering society or an engineering vocational guidance program.

"Where no such program exists, practicing sanitary engineers and sanitary engineering teachers should work actively to have one organized, not only for the benefit of their special line, but for the benefit of engineering and prospective engineering students in general. Where there is a program, sanitary engineers should make certain that their line of work is properly presented. They know well that it is of vital and growing importance. They see that, perhaps because of the relative smallness of the field, students do not as a rule take the simplest and most efficient means of acquiring proficiency in it. They see men, apparently born sanitary engineers, who started out as civil engineers or general chemists and only became proficient in their lifework by diligent private study after graduation. They see sanitary engineering positions inadequately filled, because of lack of men with proper fundamental training. If there is within your reach a young man who is destined to become an able sanitary engineer, you should help him, if possible, to find his lifework by the end of his first college year, rather than a year or two after graduation."

Recommendations

This Committee recommends:

(1) That steps be taken to coordinate the interests and the activities of the several organized groups of sanitary engineers, particularly of the Sanitary Engineering Division of the Society, the Engineering Section of the American Public Health Association, the Conference of State Sanitary Engineers, and the Conference of Municipal Public Health Engineers;

(2) That broader training in sanitary engineering, to include generally more chemistry, biology, sanitation, and economics, be encouraged in the technical institutions of the United States;

(3) That broader training of health officers and of the medical profession in general in respect to environmental sanitation be encouraged to the end that they shall gain a better appreciation of the entire field of public health, including that which is or rationally may be compassed by sanitary engineering. In other words, the modern medical student should receive more instruction in preventive medicine than has commonly been the case in the past;

(4) That the feasibility be explored of breaking down geographical barriers and political boundaries of states, counties, and cities with the hope of making

it possible to employ the best available sanitary engineering personnel for any and all positions, thus overcoming a handicap which now operates to discourage many able men from entering this field, prevents the filling of positions with qualified men, and retards advancement of individuals consistent with their capacity;

(5) That this Committee, or a joint committee representing the four organized groups of sanitary engineers named in Item (1), prepare a manual surveying the possible professional activities of trained sanitary engineers, including their services in the field of public health, broadly interpreted;

(6) That a mailing list be prepared of those authorities who are responsible for the education and employment of sanitary engineers in general, and particularly including those who are responsible for the personnel dealing with the more recently developed phases of environmental control properly demanding the services of sanitary engineers or the engineering approach to the successful solution of the problems presented;

(7) That the manual suggested in Item (5) be made available to all known potential and actual employers of sanitary engineers, as well as to educational institutions referred to in Item (6);

(8) That a minimum schedule of basic courses or program of work in sanitary engineering be devised for the guidance of educational institutions proposing to furnish really adequate undergraduate training therein;

(9) That a schedule of minimum salaries be set up for sanitary engineers in public service, following procedures instituted by the Board of Direction of the Society in connection with the salaries of engineers employed by certain state highway departments;[10]

(10) That stress be placed upon vocational guidance throughout the United States as related to the education and service of the sanitary engineer;

(11) That consideration be given to the devising of an appraisal form whereby sanitary engineers may evaluate the adequacy of existing regional programs of applied environmental sanitation in relation to the problems encountered, recognizing such factors as relate to the quality and extent of professional and quasi-professional service by all agencies operating in the region under investigation.

Respectfully submitted,

J. J. Bloomfield
Thomas R. Camp
Frank Clarke Dugan
Gordon M. Fair
William A. Hardenbergh
Harry E. Miller
Charles Lundy Pool
Charles Gilman Hyde, *Chairman*
Committee on Advancement of
Sanitary Engineering.

June 15, 1942

[10] "Surveys of Highway Engineering Positions and Salaries," *Manual of Engineering Practice No. 24*, Am. Soc. C. E., July 21, 1941.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FUNDAMENTAL ASPECTS OF THE DEPRECIATION PROBLEM A SYMPOSIUM

Discussion

BY MESSRS. JOHN F. MILLER, AND FRED ASA BARNES

JOHN F. MILLER,[49] ESQ.[49a]—Theories of analysis and deduction are valueless unless actuated by sound methods. In none of the papers of this Symposium does there appear any information as to a practical method of collecting requisite data to determine a sound and true depreciation rate. The most important and dominant element in valuation is depreciation, and the most important elements in depreciation, as applied to all progressive industrial organizations in the United States, are (1) obsolescence and (2) inadequacy. Other elements—wear and tear, maintenance and deterioriation—are of rather secondary importance.

The industrial developments in the United States have been so rapid since the beginning of the twentieth century that mechanically sound machinery and equipment often become useless because industry is finding new and larger, or different equipment so valuable that it has been profitable to scrap units which yesterday were considered the best. To decide the economy of such retirement, however, requires a knowledge of local conditions and a general understanding of the industry as a whole which no outside valuation expert, however proficient he may be, possesses to a degree to qualify him, alone, to establish a sound depreciation rate.

When he starts such work in a new plant, the writer has made it a practice to become acquainted with the foreman and operators of the various machines and equipment. In an informal manner he undertakes to acquaint them with the problem and to start them thinking along the lines of what they see, surmise,

NOTE.—This Symposium was published in November, 1941, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: November, 1941, by Messrs. Edwin F. Wendt and L. T. Fleming, and Anson Marston; December, 1941, by Messrs. William G. Atwood, George E. Goldthwaite, Nathan B. Jacobs, H. J. Flagg, Nelson Lee Smith, and C. Beverley Benson; February, 1942, by Messrs. Paul T. Norton, Jr., H. L. Ripley, and Carroll A. Farwell; March, 1942, by Messrs. Thomas R. Agg, Conde B. McCullough, W. V. Burnell, Roger L. Morrison, Wallace B. Carr, W. L. Waters, A. G. Mott, David A. Kosh, E. G. Walker, and K. Lee Hyder; and April, 1942, by Messrs. James T. Ryan, John C. Page, and John S. Worley.

[49] Cons. Engr., Washington, D. C.

[49a] Received by the Secretary April 6, 1942.

or dream, for the future development of the machines and equipment which they handle. For example, what improvements, and along what lines, do there seem to be indications of changes either in method, or need of manufacture, of the articles now produced? The writer explains, in layman's language, the problem of obsolescence and inadequacy (not using these terms at first without a clear understandable definition).

At first the response was negative, but as the work progressed, acquaintance with the men grew, and in the process of reviewing other similar problems the writer was gratified to note the development in the thinking of the shopmen, and the sound conclusions formed as to the future value of their machines and equipment. Men who direct the operations and operate machines for any length of time come to know the limitations of those machines and to dream of improvement.

The collection of these developed opinions, based on sober continued thought during the course of the work—their assimilation, balancing and comparing with other data of other plants—never failed to produce invaluable information which, to a rather large extent, determined the depreciation factor to be used in establishing a sound and true "present value" of the equipment of the plant.

The innumerable tables of straight-line, and other depreciation rates extant, served as a crude check, but such values were many times considerably, and sometimes radically, changed by this collection of first-hand knowledge and opinions of those whose work and experience peculiarly qualify them to have a voice in rate making. This procedure requires time and patience—patience to picture the problem to men who are unfamiliar with valuation work, particularly obsolescence (a word that rather stopped their thinking temporarily). When the picture was perfectly clear, however, the writer found a ready, intelligent response, which gave a much clearer vision of the value now existing than the most skilful experts working alone could possibly have obtained.

FRED ASA BARNES,[50] M. AM. SOC. C. E.[50a]—The central purposes of this Symposium—"To avoid emphasis on technical details and make a new beginning toward formulating a preliminary agreement on fundamentals," as stated by Mr. Scharff in his "Foreword"—seem to have miscarried badly; there are plenty of technical details in the second and third papers and nothing particularly new in any of them.

Why continue this battle of words about little or nothing? All agree that there is such a thing as depreciation, but not on how to determine it. All might agree, however, that the first essential to its determination with a fair and reasonable degree of accuracy is an honest intention to do so. A company that sets up depreciation reserves three or four times larger than necessary to meet requirements according to its own replacement records is not attempting to determine depreciation accurately but is attempting to "put something over" on the authorities and the public. A company that uses one set of depreciation figures for the Bureau of Internal Revenue and an entirely different set for the authority that governs its rates is also scarcely honest. Although

[50] Prof., Railroad Eng., School of Civ. Eng., Cornell Univ., Ithaca, N. Y.

[50a] Received by the Secretary April 30, 1942.

these and other irregularities have caused some of the existing confusion, it is becoming increasingly safe to leave them in the hands of the proper authorities whose duty it is to detect them; but much difference of opinion will still be found in regard to the main problem—how to determine depreciation.

An example to support the foregoing is as follows: Assume a machine with a first cost of $10,000, a rated life of six years, a salvage value of $400, a service capacity of 2,000 hr per yr, and annual repair costs as given in Table 1. The differences between the results determined by the straight-line and the annuity methods are seen to be comparatively small. Furthermore, the effect of the heavy repair costs in the later years is not obscured as it is with the Matheson or the constant-percentage method.

TABLE 1.—COMPARISON OF THE COSTS OF REPAIRS AND CARRYING CHARGES BY DIFFERENT DEPRECIATION METHODS
(Interest Rate, 7%)

Year	Hours	Repair costs[a] (dollars)	DOLLARS PER HOUR OF SERVICE			
			Straight line	Annuity	Matheson	Equalized
1	2,000	2,100	2.20	2.07	3.48	2.524
2	4,000	1,000	1.59	1.52	1.92	2.524
3	6,000	3,330	2.70	2.69	2.49	2.524
4	8,000	3,570	2.77	2.81	2.27	2.524
5	10,000	4,000	2.93	3.02	2.28	2.524
6	12,000	4,800	3.27	3.42	2.57	2.524

[a] Substantially as given in "Construction Planning and Plant," by A. J. Ackerman and C. H. Locher, 1st Ed., McGraw-Hill Book Co., Inc., New York, N. Y., and London, England, 1940; see Fig. 22, p. 70.

The equalized results in the last column of Table 1 were computed by using the annuity equivalent to the repair costs; but they could be approximated by using the average annual repair cost, $3,167, discounted by 5% to 7% because of the larger values in the early years due to the use of the average. The yearly depreciations or "write-offs" by this method are as follows:

Age	Depreciation (dollars)
1	2,248
2	3,505
3	1,421
4	1,280
5	940
6	206

These are quite variable, of course, but none is as large as that for the first year by the Matheson method ($4,150).

The writer firmly believes that slavishness to method is one of the major difficulties, and there is absolutely no reason for conforming to any particular method; for it can be proved easily, if any proof is necessary, that there is no financial difference between the different methods, provided that interest is paid or properly accounted for, except for the case of the sinking-fund method when the rate of interest paid on deposits in the fund differs from the rate paid on the debt. That is, the sum of the present worths of the payments actually made, whatever the method or lack of method, will be the debt, and why should it be anything other than the debt?

Thus, in many cases where life histories are available (including repair and

overhauling records), it would be possible to adjust carrying charges so that the total cost per unit of product would remain nearly constant throughout the life of the machine or structure, which would be an ideal result from the standpoint of fairness and equity. Of course, obsolescence might "upset the applecart," and in the case of construction machinery the annual or seasonal use would be most difficult to predict. In any case, "adjusting entries" might be necessary from time to time. They are used quite freely for other purposes, however, so why should they not be used for correcting depreciation estimates?

As to a definition of depreciation, the writer suggests the one formulated by Chief Justice Charles Evans Hughes on April 30, 1934, in the so-called Chicago Telephone Case. It is:

> "Broadly speaking, depreciation is the loss, not restored by current maintenance, which is due to all the factors causing the ultimate retirement of the property. These factors embrace wear and tear, decay, inadequacy and obsolescence."

Aside from the wilful and intentional misrepresentations by utilities through their lawyers and accountants, some writers on these subjects have contributed their quota to the existing confusion of thought. One very prominent author calls an annuity a depreciation payment, although the context indicates plainly his knowledge that an annuity consists of interest and depreciation. He states also that the straight-line method is the most straightforward (whatever that may mean) but that the diminishing-balance method (diminishing-value or Matheson method, to engineers) has the advantage of simplicity since no complex calculations or elaborate records are necessary. If the percentage loss is assumed (which seems probable since he gives no method of computing it), this method would still require more computations than the straight-line method, as both the depreciation and the interest change from year to year.

Professor Grant, in Example 1, complicates the problem by introducing a change in price level, and the writer does not agree that there is no "popular" depreciation. Certainly the $6,000 received for the property has less purchasing power than the $6,000 paid for it. Furthermore, the owner would have a difficult time convincing the Bureau of Internal Revenue that he was not subject to a capital gains tax on the total depreciation he had written off while the property was leased.

Another example of confused thought is a recent book in which the sinking fund is called an annuity, and the sinking-fund method is lauded because the fund accumulates at compound interest while the "accumulation of carrying charges is at simple interest." Of course, there is no "accumulation of carrying charges" if interest is paid or accounted for annually, as it should be; furthermore, the amount in the sinking fund is only a small fraction of the debt at first and reaches the amount of the debt only at the very end of the period.

Finally, the writer has no patience with those who maintain that there is a conflict between theory and practice, or actualities. Correct theory and correct practice support and sustain each other, and one or the other is wrong if there is a conflict between them.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

ALLOCATION OF THE TENNESSEE VALLEY AUTHORITY PROJECTS

Discussion

BY MESSRS. SHERMAN M. WOODWARD, AND ALBERT R. ARLEDGE

SHERMAN M. WOODWARD,[6] M. AM. SOC. C. E.[6a]—For many people the best approach to the allocation problem is from the point of view of the most reasonable distribution of the joint savings or profits in a multiple-purpose project. The data given in Table 1 are most convenient to illustrate this approach. Here a triple-purpose project for navigation, flood control, and power is assumed to cost only $1,000,000, whereas the justifiable expenditure for the three purposes, planned separately (see Col. 2, Table 1), totals $1,300,000. Thus, there is a savings or profit of $300,000 due to combining the three facilities in a joint undertaking rather than building them independently of each other. The amount saved is then allocated to each purpose so

TABLE 6.—ASSUMED ALLOCATION OF $1,000,000

Facility	Justifiable investment	Direct costs	Remaining justifiable investment[a]	Percentage of total	Allocation of joint savings	Allocated cost[b]	Percentage of total
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Navigation	$400,000	$150,000	$250,000	31.3	$94,000	$306,000	30.6
Flood control	300,000	100,000	200,000	25.0	75,000	225,000	22.5
Power	600,000	250,000	350,000	43.7	131,000	469,000	46.9
Total	$1,300,000	$500,000	$800,000	100.0	$300,000	$1,000,000	100.0

[a] Col. 2 minus Col. 3. [b] Col. 2 minus Col. 6.

as to reduce the amount of the investment charged to each, as illustrated in Table 6. The final allocated cost values shown in Table 6 are the same as in Table 1.

NOTE.—This paper by Theodore B. Parker, M. Am. Soc. C. E., was published in December, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: February, 1942, by Malcolm Elliott, M. Am. Soc. C. E.; March, 1942, by E. L. Chandler, M. Am. Soc. C. E.; and May, 1942, by Eugene L. Grant, M. Am. Soc. C. E.

[6] Chf. Water Control Planning Engr., TVA, Knoxville, Tenn.

[6a] Received by the Secretary May 15, 1942.

The allocation of joint costs seems to be a matter of accounting rather than of economics. It resembles somewhat the making of appraisals—a common procedure in connection with condemnation cases, bankruptcy cases, inheritance cases, insurance settlements, and damage cases, as well as in commercial transactions of all kinds. It requires knowledge, experience, and judgment in making the voluminous computations.

In the method of allocation used in Table 2, if one takes the difference between the columns for the seven-plant system and the six-plant system in an attempt to find the effect of the Hiwassee project, Items 2 and 10 seem to show that the justifiable investment in Hiwassee for the benefit of navigation is $1,000,000, whereas Items 14 and 18 seem to show that the actual computed allocation of the Hiwassee investment to navigation is $1,674,439. Of course,

TABLE 7.—System Allocations (Alternate Method)
(Amounts in Dollars Are Given to the Nearest Hundred Thousand)

No.	Facility	Six-Plant System, Including Chickamauga Dam		Seven-Plant System, Including Hiwassee Dam		Col. 5 Minus Col. 3		Effect of Hiwassee Dam	
		Dollars	%	Dollars	%	Dollars	%	Dollars	%
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	Total investment allocated	1,933	....	2,103	....	170	....	...	....
	Justifiable Investment:								
2	Navigation	1,052	....	1,062	....	10	....	...	....
3	Flood control	523	....	633	....	110	....	...	....
4	Power	1,389	....	1,500	....	111	....	...	....
5	Total	2,964	....	3,195	....	231	....	...	....
	Direct Costs:								
6	Navigation	172	....	172	....	...	....	...	....
7	Flood control	47	....	49	....	2	....	...	....
8	Power	531	....	563	....	32	....	...	....
9	Total	750	....	784	....	34	....	...	....
	Remaining Justifiable Investment:								
10	Navigation	880	39.7	890	36.9	10	5.1	...	....
11	Flood control	476	21.5	584	24.2	108	54.8	...	....
12	Power	858	38.8	937	38.9	79	40.1	...	....
13	Total	2,214	100.0	2,411	100.0	197	100.0	...	....
	Allocation of Common Costs:								
14	Navigation	470	39.7	487	36.9	17	12.5	7	5.1
15	Flood control	254	21.5	319	24.2	65	47.8	75	54.8
16	Power	459	38.8	513	38.9	54	39.7	54	40.1
17	Total	1,183	100.0	1,319	100.0	136	100.0	136	100.0
	Allocation of Total Costs:								
18	Navigation	642	33.2	659	31.3	17	10.0	7	4.1
19	Flood control	301	15.6	368	17.5	67	39.4	77	45.3
20	Power	990	51.2	1,076	51.2	86	50.6	86	50.6
21	Total	1,933	100.0	2,103	100.0	170	100.0	170	100.0

it was not intended that Table 2 should be used in this way, but if any one thinks that this is an important question, it is a simple matter to modify slightly the method of making the computation and remove the apparent inconsistency, as is illustrated in Table 7.

This table includes the first 21 items for part of the columns of Table 2. To simplify the presentation, all of the amounts stated in dollars have been changed to the nearest hundred thousand and then the last five ciphers are omitted. Col. 7, obtained by subtracting Col. 3 from Col. 5, shows that the total investment in Hiwassee to be allocated is $17,000,000; the total justifiable investment is $23,100,000; and the total direct costs amount to $3,400,000. This leaves $13,600,000 of common costs to be allocated, whereas the total remaining justifiable investment is $19,700,000. Col. 9 shows that if the common costs in Hiwassee are allocated in proportion to the components of the remaining justifiable investment in the Hiwassee project, finally the total investment in the Hiwassee project would be allocated approximately as follows: $700,000 to navigation, $7,700,000 to flood control, and $8,600,000 to power. Compared with the results in Col. 7, the figures in Col. 9 show about $1,000,000 less charged to navigation, $1,000,000 additional charged to flood control, and no change in the charge to power.

ALBERT R. ARLEDGE,[7] M. AM. SOC. C. E.[7a]—Mr. Parker has made an interesting and useful presentation of the theories of allocation for general application as well as specifically for the TVA. The writer believes that many engineers have been puzzled as to how the various costs were determined for each element in a multiple-purpose project. This paper, the writer feels, has been helpful in clearing away the mist that formerly hung over such projects.

Under the heading "Other Similar Allocations," the author includes Boulder Power Plant. The writer is interested in presenting a fuller and clearer picture of this project in regard to allocations. A thorough treatment of this subject would be too long and involved. Consequently, only a few of the selected high lights that bear on the financing will be treated.

The Boulder Canyon project and other major multiple-purpose projects are fairly similar in the purposes for which they were constructed—that is, to improve navigation, control floods, store water for irrigation and domestic supply, generate electrical energy, etc.—and in allocations made for the different functions. They are dissimilar when it comes to the financing or repayment to the government of the cost of the various projects. In different localities different functions receive emphasis. For example, the flood control and water conservation features were emphasized in the publicity advocating the passage of the Boulder Canyon Project Act, but power was selected to make the project "a self-supporting and financially solvent undertaking."

The principal reason for the lack of uniformity in public policy regarding these large projects is probably the fact that the Boulder Canyon Project Act was passed in December, 1928, before the Depression and the great wave of unemployment swept the United States, whereas the other projects were authorized subsequently to this crisis in a constructive attempt to reduce unemployment and improve labor conditions. In any event, between the time of enactment of the Boulder Canyon Project Act and the setting up of other multiple-purpose projects, the Congress changed its public policy from that of

[7] Prin. Civ. Engr., Los Angeles Bureau of Power & Light, Los Angeles, Calif.

[7a] Received by the Secretary May 18, 1942.

requiring self-liquidation to that of granting financial aid by allocating federal funds to functions other than power. These funds ranged from 67.5% to 43% of the ultimate cost of the Bonneville project and from 60% to 48% of the ultimate cost in the case of the TVA. Power is not obligated to repay such allocations. Power at Boulder Dam is obligated to repay all costs of the project, including the allocation to flood control.

Attention is directed to the fact that the Congress of the United States set public policy by the terms of the Boulder Canyon Project Act,[8] approved December 21, 1928, when it authorized the Secretary of the Interior, in Section 1, in part, as follows:

> "* * * for the purpose of controlling the floods, improving navigation and regulating the flow of the Colorado River, providing for storage and for the delivery of stored waters thereof for the reclamation of public lands and other beneficial uses exclusively within the United States, and for the generation of electrical energy as a means of making the project herein authorized a self-supporting and financially solvent undertaking, * * * subject to the terms of the Colorado River Compact herein mentioned * * * to construct, operate, and maintain a dam and incidental works in the main stream of the Colorado River at Black Canyon or Boulder Canyon adequate to create a storage reservoir of a capacity of not less than twenty million acre-feet of water * * *: Provided * * * also to construct and equip, operate, and maintain at or near said dam, or cause to be constructed, a complete plant and incidental structures suitable for the fullest economic development of electrical energy from the water discharged from said reservoir * * *."

Although the foregoing section authorized the Secretary of the Interior to construct the Boulder Canyon project, it was necessary to meet the further provision of financing before work could proceed legally. This provision follows:

> "Sec. 4(b). Before any money is appropriated for the construction of said dam or power plant, or any construction work done or contracted for, the Secretary of the Interior shall make provision for revenues by contract, in accordance with the provisions of this Act, adequate in his judgment to insure payment of all expenses of operation and maintenance of said works incurred by the United States and the repayment, within fifty years from the date of the completion of said works, of all amounts advanced to the fund under subdivision (b) of Section 2 for such works, together with interest thereon made reimbursable under this Act." [8]

The "Project Act" makes an allocation for flood control purposes. C. C. Elder,[9] M. Am. Soc. C. E., has given the background of this project in greater detail than can be given herein. To quote Mr. Elder on this subject:

> "All advances to the Colorado River Dam fund from the U. S. Treasury were required to be repaid with four per cent interest, but a definite allocation of $25,000,000 was made to flood control to be repaid out of 62.5 per cent of excess revenues, if and when available. If this repayment were not completed within the general 50-year amortization period, ending May 31, 1987, repayment was to be continued out of the same proportion

[8] "Boulder Canyon Project Act," *Public No. 642*, H. R. 5773, 70th Cong., 2d Session.

[9] "Boulder Canyon Project, Original and Readjusted Repayment Contracts," by C. C. Elder—presented before the Engineering Economics Division, Am. Soc. C. E., 89th Annual Meeting, New York, N. Y., January 22, 1942.

of the project's net revenues. Thus deferment was permissible for the flood control allocation's repayment, if this became necessary, but provision was made for full repayment finally, with four per cent compound interest. The amount of this flood control allocation appears to have been fixed arbitrarily, so far as is now evident, but is approximately the sum that might have been needed for a program of flood control by means of a low dam for this purpose solely, and a system of raised and strengthened levees. This solution would have given immediate, but evidently only temporary, relief from flood damages, due to relatively rapid filling with silt of a small reservoir. But the federal obligation to furnish some degree of flood control on the lower Colorado River was recognized, and this is probably the only case to date where repayment, even though possibly on a deferred basis, of a government flood control structure, or its allocated share of the construction cost, has ever been required or promised.

"In addition to the flood control allocation, other excess revenues of the Colorado River Dam fund were allotted as follows:

> " 'If during the period of amortization the Secretary of the Interior shall receive revenues in excess of the amount necessary to meet the periodical payments to the United States as provided in the contracts executed under this act, then, immediately after the settlement of such periodical payments, he shall pay to the State of Arizona 18¾% of such excess revenues and to the State of Nevada 18¾% of such excess revenues.
>
> " 'After the repayments to the United States of all money advanced with interest, charges shall be on such basis and the revenues derived therefrom shall be kept in a separate fund to be expended within the Colorado River Basin as may hereafter be prescribed by the Congress.'

"Records of Congressional Committee hearings indicate that the intent of Congress was that Arizona and Nevada should receive some payments in lieu of taxes which would have been levied if the project had been privately developed. The amount is stated as having been arrived at on the basis of a fairly parallel case, the leasing of public oil lands, where the rule had become established that the State involved should receive 37 per cent of the net revenue in lieu of taxes. The disposition of the balance of excess revenues, after completion of repayment of U. S. Treasury advances to the fund, was such a remote problem as not to excite immediate interest, so that it was merely left to future action of Congress, though allotted for Colorado River Basin developments if and when available."

In compliance with the foregoing authorization and provisions, the Secretary of the Interior, after investigation and study of regional market conditions, set a schedule of rates in Article (16) of the contract, dated April 26, 1930, a part of which follows:

> "(16) In consideration of this lease, the lessees severally agree:
>
> "(1) To pay the United States for the use of falling water for the generation of energy for their own use, respectively, by the equipment leased hereunder (except as otherwise provided in Article Seventeen (17) hereof), as follows:
>
> "(*a*) One and sixty-three hundredths mills ($0.00163) per kilowatt-hour (delivered at transmission voltage) for firm energy;
>
> "(*b*) One-half mill ($0.0005) per kilowatt-hour (delivered at transmission voltage) for secondary energy; * * *.

> "At the end of fifteen (15) years from the date of execution of this contract and every ten (10) years thereafter, the above rates of payment for firm and secondary energy shall be readjusted upon demand of any party hereto, either upward or downward as to price, as the Secretary (of Interior) may find to be justified by competitive conditions at distributing points or competitive centers." [10]

Based on the rates established in the aforementioned contracts, prepared and executed under the original act, the two lessees, the Los Angeles Municipal Bureau of Power and Light and the Southern California Edison Company, Ltd., assumed the financial responsibility for the return of the full costs of the dam, power house, and incidental structures in 50 years at 4% interest, and for the return of the cost of machinery and equipment in ten annual installments, as set up in Article (9)(a) of the contract which, in part, follows:

> "(9)(a) Compensation for the use, for the periods of lease thereof, of machinery and equipment furnished and installed by the United States, for each lessee respectively, for the generation of electrical energy, equal to the cost thereof, including interest charges at the rate of four per centum (4%) per annum, compounded annually from the date of advances to the Colorado River Dam Fund for the purchase of such equipment and machinery to June 1st of the year next preceding the year when the initial installment becomes due under this article, shall be paid to the United States by the lessees, severally, in ten (10) equal annual installments, so as to amortize the total cost (including interest as fixed above), and interest thereafter upon such total cost at the rate of four per centum (4%) per annum * * *." [10]

The power obligation of these two lessees totaled 64% of the firm power (this included the 18% allotment to each of the states of Arizona and Nevada, divided equally between the two lessees). The states assumed no financial responsibility in regard to the capital outlay. At that time, the Metropolitan Water District of Southern California, whose allotment was 36% of the Boulder Dam firm energy, was not considered a going concern, as the bonds for the construction of its Colorado River Aqueduct had not yet been voted.

From the preceding sketchy review of the requirements and provisions of the Project Act and the contracts, and the additional fact that the Metropolitan Water District long since had voted its bonds and had become a going concern, it would seem apparent that a revision of the contracts was necessary to maintain justice and equity between the government and the lessees.

The Boulder Canyon Project Adjustment Act[11] permits the contracts for energy to be adjusted to rates more comparable with the liberal public policy adopted by the Congress for other multiple-purpose projects throughout the United States. It recognized 100% power allotment for amortization purposes instead of 64% of the allotment as set in the original contracts. The "Adjustment Act" also provides more certain payments to the states of Arizona and Nevada and to the Colorado River Development Fund.

The Secretary of the Interior was interested, from an administration stand-

[10] Contract for Generation and Transmission of Power, Boulder Canyon Project—Bureau of Reclamation, U. S. Dept. of the Interior; the City of Los Angeles, Calif.; and Southern California Edison Company, Ltd., April 26, 1930.

[11] "The Boulder Canyon Project Adjustment Act," approved July 19, 1940, *Public No.* 756, H. R. 9877, 76th Cong., 3d Session.

point, in changing the lessee from a "Contract for Lease of Power Privilege" to that of an "Agent of the United States for the Operation of the Power Plant." The Adjustment Act provided the mechanics under which the desirable changes suggested could be made. The adjusted rates for Boulder Dam energy will more nearly meet "competitive conditions at distributing points or competitive centers," will still comply with the rigorous requirements of the original Act for a self-supporting project, and will reduce the initial rates roughly 30%.

Under "Limits of Variation" the author has well stated:

> "Viewing this problem in its broadest sense, it is evident that no allocation can be determined with mathematical precision. On the other hand, there are reasonable limits to the variations in allocations of common cost.
>
> "Where three major purposes are involved, it would be an obvious absurdity to charge more than half of the common cost to power alone, especially where power production is designated by congressional legislation as incidental to the other two purposes. On the other hand, where power is the only 'paying partner,' it is difficult to justify charging to power less than one third of the common costs in any event."

The writer feels that the author, in these two paragraphs, has expressed a policy in regard to allocations which could well be adopted by the Congress as a guide to simplicity and uniformity in public policy for the benefit of all multiple-purpose projects that it initiates.

In summarizing, the Adjustment Act has effected changes, as follows:

1. Lessee to operating agent;
2. 4% interest to 3% interest;
3. (*a*) The rate for firm energy from 1.63 mills per kw-hr to 1.163 mills per kw-hr; and (*b*) the rate for secondary energy from 0.50 mill per kw-hr to 0.34 mill per kw-hr;
4. Payment for machinery and equipment from ten annual installments to 50 years;
5. The postponement of the payment of the flood control allocation without interest until the end of the 50-yr amortization period, May 31, 1987;
6. The payments of 18¾% of

> "* * * revenues in excess of the amount necessary to meet the periodical payments to the United States as provided in the contract, or contracts, executed under this Act * * *" [8]

if any, to each of the states of Arizona and Nevada, to the payment

> "* * * to each of said States of the sum of $300,000 for each year of operation, beginning with the year of operation ending May 31, 1938, and continuing annually thereafter until and including the year of operation ending May 31, 1987 * * *" [11]; and

7. The payment for the Colorado River Basin development changed from the stipulation that

> "* * * after the repayment to the United States of all money advanced with interest, charges shall be on such basis and the revenues derived

> therefrom shall be kept in a separate fund to be expended within the Colorado River Basin as may hereafter be prescribed by the Congress," [8]

to a more assured and earlier payment, as defined by:

> "* * * the sum of $500,000 for the year of operation ending May 31, 1938, and the like sum of $500,000 for each year of operation thereafter, until and including the year of operation ending May 31, 1987 * * *." [11]

Items 6 and 7 are subject to a further provision as set forth in Section 3 of the Adjustment Act, as follows:

> "If, by reason of any act of God, or of the public enemy, or any major catastrophe, or any other unforeseen and unavoidable cause, the revenues, for any year of operation, after making provision for costs of operation, maintenance, and the amount to be set aside for said year for replacements, should be insufficient to make the payments to the States of Arizona and Nevada and the transfers to the Colorado River Development Fund herein provided for, such payments and transfers shall be proportionately reduced, as the Secretary may find to be necessary by reason thereof." [11]

An unusual feature of the Adjustment Act is its retroactive character, requiring these changes to become effective June 1, 1937.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

DRAINAGE OF LEVEED AREAS IN MOUNTAINOUS VALLEYS

Discussion

BY W. B. LANGBEIN, ASSOC. M. AM. SOC. C. E.

W. B. LANGBEIN,[18] ASSOC. M. AM. SOC. C. E.[18a]—Any treatment of the flood problem even though directed to one specific design is likely to involve the application of a large range of hydrologic principles. The designer of drainage structures or leveed areas finds it necessary not only to familiarize himself with, but also to evaluate, rainfall and snow-melt characteristics, the absorptive qualities of the ground and the laws of open-channel flow, besides the hydraulic and mechanical properties of the drainage structures themselves. There is special need, therefore, for methods of solving the many hydrologic questions that arise in the design of hydraulic structures and are common to most flood and even drought problems. Methods of treatment in one of them merit consideration in another, and Mr. Williams' paper, although perhaps limited in title, contributes much to this end.

The writer was most interested in Mr. Williams' construction of flood discharge hydrographs from known or assumed amounts of effective rainfall and snow melt. This feature of the rainfall-runoff relation has been a favorite subject for some years; and with increasing experience, fairly satisfactory results have been attained, mainly through the unit hydrograph, a hydrologic tool which reaches its tenth anniversary this year (1942). However, there is noted in Mr. Williams' treatment a co-mingling of the so-called rational method of earlier origin with a synthetic unit-graph method. At the possible risk of confusing the subject, the writer offers the following method for solving Mr. Williams' problem, which breaks away entirely from the so-called rational procedure, substitutes some of the newer unit-graph technique and flood-routing principles, and yet is believed to be as direct and simple as the method used by Mr. Williams.

NOTE.—This paper by Gordon R. Williams, Assoc. M. Am. Soc. C. E., was published in January, 1942, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1942, by Merrill Bernard, M. Am. Soc. C. E.; April, 1942, by L. K. Sherman, M. Am. Soc. C. E.; and May, 1942, by Messrs. C. O. Clark, W. W. Horner, and Walter T. Wilson.

[18] Associate Hydr. Engr., U. S. Geological Survey, Washington, D. C.

[18a] Received by the Secretary April 21, 1942.

> therefrom shall be kept in a separate fund to be expended within the Colorado River Basin as may hereafter be prescribed by the Congress," [8]

to a more assured and earlier payment, as defined by:

> "* * * the sum of $500,000 for the year of operation ending May 31, 1938, and the like sum of $500,000 for each year of operation thereafter, until and including the year of operation ending May 31, 1987 * * *." [11]

Items 6 and 7 are subject to a further provision as set forth in Section 3 of the Adjustment Act, as follows:

> "If, by reason of any act of God, or of the public enemy, or any major catastrophe, or any other unforeseen and unavoidable cause, the revenues, for any year of operation, after making provision for costs of operation, maintenance, and the amount to be set aside for said year for replacements, should be insufficient to make the payments to the States of Arizona and Nevada and the transfers to the Colorado River Development Fund herein provided for, such payments and transfers shall be proportionately reduced, as the Secretary may find to be necessary by reason thereof." [11]

An unusual feature of the Adjustment Act is its retroactive character, requiring these changes to become effective June 1, 1937.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

DRAINAGE OF LEVEED AREAS IN MOUNTAINOUS VALLEYS

Discussion

BY W. B. LANGBEIN, ASSOC. M. AM. SOC. C. E.

W. B. LANGBEIN,[18] ASSOC. M. AM. SOC. C. E.[18a]—Any treatment of the flood problem even though directed to one specific design is likely to involve the application of a large range of hydrologic principles. The designer of drainage structures or leveed areas finds it necessary not only to familiarize himself with, but also to evaluate, rainfall and snow-melt characteristics, the absorptive qualities of the ground and the laws of open-channel flow, besides the hydraulic and mechanical properties of the drainage structures themselves. There is special need, therefore, for methods of solving the many hydrologic questions that arise in the design of hydraulic structures and are common to most flood and even drought problems. Methods of treatment in one of them merit consideration in another, and Mr. Williams' paper, although perhaps limited in title, contributes much to this end.

The writer was most interested in Mr. Williams' construction of flood discharge hydrographs from known or assumed amounts of effective rainfall and snow melt. This feature of the rainfall-runoff relation has been a favorite subject for some years; and with increasing experience, fairly satisfactory results have been attained, mainly through the unit hydrograph, a hydrologic tool which reaches its tenth anniversary this year (1942). However, there is noted in Mr. Williams' treatment a co-mingling of the so-called rational method of earlier origin with a synthetic unit-graph method. At the possible risk of confusing the subject, the writer offers the following method for solving Mr. Williams' problem, which breaks away entirely from the so-called rational procedure, substitutes some of the newer unit-graph technique and flood-routing principles, and yet is believed to be as direct and simple as the method used by Mr. Williams.

NOTE.—This paper by Gordon R. Williams, Assoc. M. Am. Soc. C. E., was published in January, 1942, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1942, by Merrill Bernard, M. Am. Soc. C. E.; April, 1942, by L. K. Sherman, M. Am. Soc. C. E.; and May, 1942, by Messrs. C. O. Clark, W. W. Horner, and Walter T. Wilson.

[18] Associate Hydr. Engr., U. S. Geological Survey, Washington, D. C.

[18a] Received by the Secretary April 21, 1942.

Fig. 7 shows standard dimensionless unit hydrographs based on a dimensionless S-hydrograph previously derived by the writer.[19] The scale is expressed in terms of the lag between center of mass of effective rainfall and flood runoff, a constant or near constant for any drainage basin. The lag has three other corollary meanings: (1) It is equal to the quotient between an increment in storage in the basin and the corresponding increment in discharge; thus it represents the storage element, and is equivalent to the flood-routing coefficient K;[20] (2) it is equal to a weighted mean time of wave travel from the

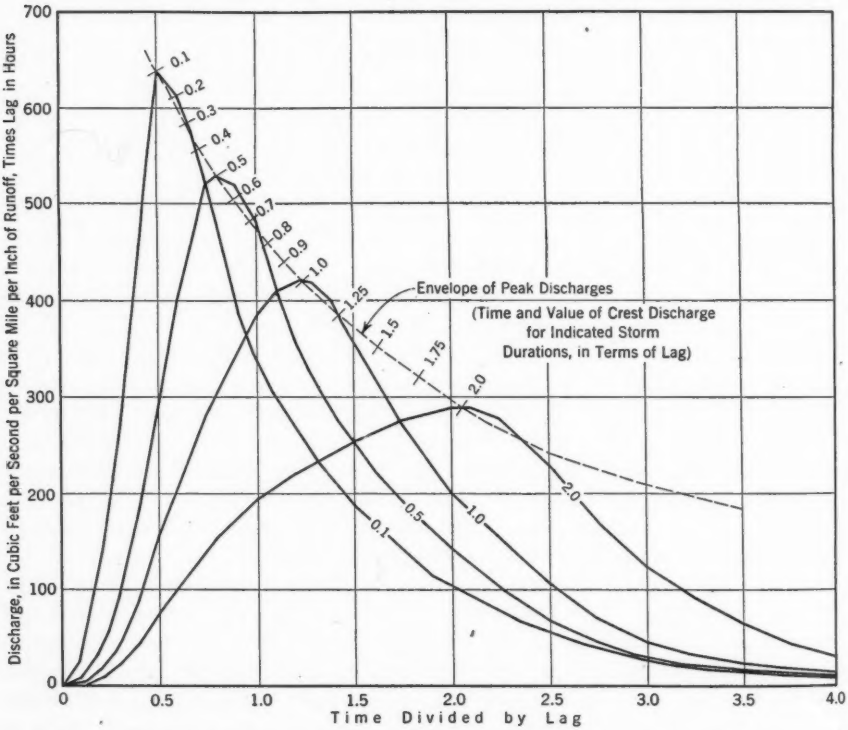


FIG. 7.—DIMENSIONLESS DIAGRAM SHOWING FAMILY OF STANDARD UNIT HYDROGRAPHS FOR SEVERAL SELECTED DURATIONS OF RUNOFF GENERATION

several component parts of the basin; and (3) it is equal to the storage coefficient as derived from recession limbs of flood hydrographs. It is computed by either comparing times of center of mass of effective rainfall and center of mass of direct runoff, or by analysis of the recession curve.[21] The lag thus defined and computed is descriptive of the storage and concentration char-

[19] "Channel Storage and Unit Hydrograph Studies," by W. B. Langbein, *Transactions*, Am. Geophysical Union, 1940, p. 626.

[20] "The Unit Hydrograph and Flood Routing," by G. T. McCarthy, unpublished ms. presented at Conference of North Atlantic Div., Corps of Engrs., U. S. Army, June 24, 1938.

[21] "Channel Storage and Unit Hydrograph Studies," by W. B. Langbein, *Transactions*, Am. Geophysical Union, 1940, p. 625.

acteristics of a basin and may be correlated with basin topographic characteristics in ways dependent on local conditions. Generalizations cannot be given; in some regions, good correlations can be obtained simply with length of a basin; in others, the slope or altitude needs to be introduced. F. F. Snyder,[22] Jun. Am. Soc. C. E., has published data applicable to Pennsylvania, and G. T. McCarthy,[20] Assoc. M. Am. Soc. C. E., has derived a method applicable to the Connecticut River Basin. With such correlations a unit hydrograph for application to ungaged areas within a limited region can be approximated. The procedure then is as follows: Suppose a basin of 3 sq miles has a lag of 1.67 hr and it is desired to compute the peak discharge using rainfall of critical duration conforming to the design storms given in Fig. 2. For a 10-yr storm the amounts of the rainfall plus snow melt in specified periods of time are given in Col. 3 of Table 6. Col. 2 of this table contains the time scale expressed non-dimen-

TABLE 6.—COMPUTATION OF FLOOD-PEAK DISCHARGES FOR STORM PERIODS OF SELECTED DURATIONS

Time		Volume of supply (in.)	× Peak of graph (Fig. 7) (csm-hr per in.)[a]	÷ Lag (hr)	= Peak discharge[a] Q_p
Minutes	Lag (%)				
(1)	(2)	(3)	(4)	(5)	(6)
0	0	0	...	...	...
30	0.30	0.55	585	1.67	193
60	0.6	0.75	500	1.67	224
90	0.9	0.92	440	1.67	242
120	1.2	1.05	390	1.67	245 = Max.
150	1.5	1.14	350	1.67	239
180	1.8	1.22	315	1.67	230
210	2.1	1.29	280	1.67	216
240	2.4	1.35	250	1.67	202

[a] Cubic feet per second per square mile.

sionally in terms of the basin lag (100 min in this example). The peak of unit graphs of various durations of inflow (in terms of lag) is given by the envelope curve of Fig. 7. The corresponding values are entered in Col. 4, Table 6. It will be noted that for lengthening storm durations the volume of supply increases, whereas the peak of a unit hydrograph decreases. What is the critical duration of the given storm-rainfall-characteristics of this basin? The answer is found in Col. 6, which is the product of Cols. 3 and 4. A storm of 120-min duration gives the maximum peak of this basin. The formula is thus:

$$Q_p = S\frac{\text{Vol}}{\text{Lag}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11a)$$

or

$$Q = S\frac{\text{Vol}}{\text{Lag}}A \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11b)$$

in which Q_p is the discharge in cubic feet per second per square mile, Q is the discharge in cubic feet per second, S is the ordinate of Fig. 7, and A is drainage

[22] "Synthetic Unit-Graphs," by F. F. Snyder, *ibid.*, 19th Annual Meeting, Pt. 1, 1938, pp. 447–454.

area in square miles. A unit hydrograph for the critical duration can then be read from the family of graphs given in Fig. 7.

The writer proposes the lag time as a measure of the time-discharge characteristics of drainage basins in lieu of the so-called time of concentration. The time of concentration is a concept that has not been, and seemingly cannot be, defined accurately. For example, the usual definition is the "time required for rainfall excess to flow from the most remote part of the basin to the gaging station." Mr. Williams adapts another to his problem—the time required for runoff to equal the rate of rainfall excess. Still another is the time between beginning of the flood rise and its peak. It has never been demonstrated that these are equivalent. Incidentally, C. E. Ramser, M. Am. Soc. C. E., used the latter meaning in determining time of concentration which Mr. Kirpich used in his article, to which Mr. Williams referred.[7] In discussing Mr. Kirpich's article,[23] the writer stated that the correlation presented was equivalent to a velocity of 4 ft per sec and the inclusion of the slope factor was not well supported by the available evidence. The lag time, however, can be computed in more than one way, each of which is theoretically equivalent subject only to errors or limitations of data.

Eq. 11*b* is similar in form to Eq. 4 if the latter is multiplied by the volume of runoff, except that lag is used to evaluate the storage effect and no assumption need be made as to the critical length of the storm. It is proposed for use instead of the "rational" formula $Q = C\,i\,A$. Insufficient data are given by Mr. Williams to compare the results obtained by the two methods, except to the following extent:

Fox Hollow Run, according to Fig. 4, has a lag (center of mass of supply to center of mass of hydrograph) of 20 min.

In dimensionless terms the peaks of the hydrographs shown are as follows:

(1) "Ten-minute rainfall period" = 0.50 lag; $\dfrac{1{,}300\ \dfrac{\text{ft}^3}{\text{sec-in.}}}{\dfrac{548}{640}\ \text{miles}^2} \times 0.33$ hr (= lag) = 500 (no units). The envelope curve in Fig. 7 for inflow period of 0.5 lag gives 530.

(2) "Thirty-three-minute rainfall period" = 1.67 lag; $\dfrac{1{,}000\ \dfrac{\text{ft}^3}{\text{sec-in.}}}{\dfrac{548}{640}\ \text{miles}^2} \times 0.33$ hr (= lag) = 388. The envelope curve in Fig. 7 for inflow period of 1.67 lag gives 330.

[7] *Civil Engineering*, June, 1940, p. 362.
[23] *Ibid.*, August, 1940, p. 533.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

PROTECTIVE AND REMEDIAL MEASURES FOR SANITARY AND PUBLIC HEALTH ENGINEERING SERVICES

Discussion

BY MESSRS. JAMES H. LE VAN, CLARKE GARDNER, MICHAEL J. BLEW, AND GILBERT H. DUNSTAN

JAMES H. LE VAN,[10] ASSOC. M. AM. SOC. C. E.[10a]—The Progress Report is an excellent manual offering concise information and advice to agencies engaged in Sanitary and Public Health Engineering services during wartime. Mr. Greeley is correct in stating that the water works is the most important service to be protected from any form of attack.

It cannot be too strongly emphasized that human beings will seek water to drink when they are thirsty, and many of them will not even attempt to ascertain if the water they are about to drink is safe. The writer believes that insufficient stress has been laid upon making available an emergency or alternate supply, although there is mentioned a possible emergency source "limited to a sufficient quantity for drinking purposes distributed by tank trucks" (see heading "14. Water Treatment Plants"). The importance of this emergency means of water supply and distribution cannot be overestimated.

In two instances it is stated that mobile filter units mounted on trucks are obtainable that will draw water from wherever it is available. It is very likely that these mobile water treatment plants will become available in the larger cities where they will be purchased or manufactured. However, most of the smaller cities never will have such a piece of equipment. In an emergency, where a distribution system has been damaged badly but potable water still can be obtained from the filtration plant, well supplies, or private supplies, potable water distributed by tank trucks will furnish drinking water and will be accepted readily.

NOTE.—This Progress Report of the Sanitary and Public Health Engineering Division of the National Committee of the Society, on Civilian Protection in War Time, was published in January, 1942, *Proceedings*. Discussion on this Report has appeared in *Proceedings*, as follows: March, 1942, by Donald M. Baker, M. Am. Soc. C. E.; and April, 1942, by Messrs. J. E. Burchard and F. J. Wilson, Charles Haydock, and John E. Kiker, Jr.

[10] Passed Asst. San. Engr., U. S. Public Health Service; Asst. to Liaison Officer, 4th Corps Area Headquarters, Atlanta, Ga.

[10a] Received by the Secretary March 30, 1942.

While a tank truck is being filled with water, sufficient hypochlorite solution should be added so that there will be a chlorine residual when water is drawn from the tank at its delivery point. A large number of selected individuals should be trained to prepare a hypochlorite solution, and to disinfect small amounts of water for drinking purposes in case the public supply should be entirely disrupted. In order to be ready for emergency sterilization of the water supply and of water mains, chlorine and hypochlorite should be available. This is important because chlorine is an essential war chemical.

Where there are several communities not very far apart, their resources should be pooled because of the likelihood that not any one small community will have large resources of any kind. The resources to be pooled include replacements, extra parts, repair equipment, reserve chemicals, etc. This will avoid communities duplicating equipment. Engineering services and labor also could be pooled. Adjacent towns can make arrangements so that they can be interconnected quickly in order to furnish emergency water service to each other.

Because the regular communication system may be damaged along with water supply and sewerage systems, an emergency communication system should be made ready. Short-wave radio (the police department's station), runners, bicyclists, and motorcyclists can be used. An accurate address list of all officials and employees is necessary and their whereabouts should be known twenty-four hours of each day in order that they will be ready for instant dispatch to duty. This list should be examined carefully so that any one who has given evidence of subversive inclination can be discharged. As employees of draft age are inducted into the military service, they should be replaced with tried and trusted employees from other municipal departments who have had some years of service during which their conduct has been observed.

The foregoing remarks are the writer's personal opinions and in no way are related to his official position.

CLARKE GARDNER,[11] ASSOC. M. AM. SOC. C. E.[11a]—Difficulties encountered by water works engineers and superintendents in providing effective protection to existing water-plant equipment are diverse and often complicated. In the hope that it may contribute to the clarification of this subject, undertaken by the National Committee of the Society on Civilian Protection in War Time, the writer wishes to cite selected items from a report made by him for a community with a population of 13,000.

In reviewing the past experiences of other countries, it has been noted that enemy bombers usually attempt the simultaneous destruction of vital communication centers, power stations, and public water supplies, which generally include dams, reservoirs, pumping stations, tanks, and wells. This method of procedure is natural because the enemy wishes to create confusion, slow down production, and cause widespread fires that cannot be checked easily if the water supply is materially damaged.

[11] City Engr., Salisbury, Md.

[11a] Received by the Secretary April 10, 1942.

An investigation revealed that the existing lake and river supply for Salisbury, Md., may suffice under certain conditions, but from the standpoint of furnishing potable water this does not appear to be practicable except in extreme emergencies. It is true that fire-fighting equipment or portable gasoline engines could be located near lakes or rivers and the water pumped through fire hydrants adjacent to these areas into the distribution system. This method may afford a minimum supply for fire-fighting purposes; however, several units of emergency chlorine apparatus and a large reserve supply of chlorine must be available to make the water safe for human use. The disadvantages of using fire equipment for this purpose are apparent because such equipment will be needed in several places at the same time for the purpose of combating fires arising from incendiary bombs. The fire department would be operating under a serious handicap if equipment was withdrawn to supply water from lakes or rivers to pump into the distribution system.

The provision of auxiliary supplies through driven wells in underground stations appears to be more practicable for several reasons. It is well known that moderate quantities of water can be developed in Salisbury and the immediate vicinity through the medium of wells driven to depths ranging from 60 ft to 90 ft. Five or six 10-in. wrought-iron casing wells with approximately 20 ft of strainer per well—installed at locations tentatively designated as *A*, *B*, *C*, *D*, and *E* on the "Confidential Map"—should provide between 2,000 and 2,500 gal of water per minute. Because of the fact that static water level will vary from 20 ft to 25 ft below an average ground elevation of +30.00 above sea level, it is possible to construct concealed underground stations utilizing horizontal centrifugal pumps employing "flooded suctions." These units could be connected to the existing water system with belt-driven, used automobile engines supplying the power at the pumps. To provide an average of 40 lb of curb pressure, 20-hp engines should be capable of discharging between 300 gal per min and 400 gal per min per well against a total head of 150 ft. The advantages of the underground stations are as follows:

(1) Exact location of units could not be determined by enemy bombers;
(2) Water supply obtained would be safe for consumers;
(3) Entrance could be made through manhole vaults or tunnels;
(4) No priming of pumps would be necessary because of "flooded suctions";
(5) Five or six units pumping into 45 miles of distribution system at scattered points would assure a minimum supply with satisfactory pressure;
(6) Pumps would function regardless of power failure, which might ensue during a bombing raid;
(7) Venting arrangements for exhaust gases can be made through pipes drawn to the surface;
(8) The depth of stations would be unlikely to be affected by near-by bomb explosions;
(9) Gasoline storage could be provided in near-by underground tanks;
(10) Underground stations would be economical in first cost, being much cheaper than the provision of full protection to existing plant and equipment;
(11) Used auto engines could be used—pumps would not require motors, which would eliminate priority difficulties; and

(12) Stations would be small, of brick construction with reinforced top, and built entirely underground.

In attempting to analyze the various methods of furnishing complete protection to the existing plant and water supply, the destructive effect of demolition bombs must be considered. A review of the following tabulation will give some idea of the terrific explosive force of certain types of demolition bombs:

No.	Weight (lb)	Destructive area (sq ft)
1	1,000	380
2	500	500
10	100	885

As a rule, the bomb lands at an angle, and the explosion produces a crater and discharges steel splinters and debris for a distance depending upon the bomb's explosive power. Following the explosion, a terrific pressure is built up, quickly followed by a suction wave. Both the blast and suction have an enormous destructive effect. If the space is confined, windows will blow out and walls will collapse. Simultaneously, the explosion sets up ground waves which produce a type of stress not provided for except in earthquake-proof construction. Incidentally, the strength of these waves may be sufficient to cause the collapse of near-by buildings.

From the standpoint of providing adequate protection to existing wells, pumping station building, reservoir, and valve vault, it is useless to protect any single unit without installing protective devices to safeguard all of the units herein mentioned. The protection also would have to extend to the supply mains leading from the pumping station to the distribution mains in the street and for a reasonable distance in the street where the main feeder lines are located. The over-all protection of all units is essential because these units are located practically in a group, and the possibility of one damaged unit may render all units useless to provide an adequate supply of water. This means that heavy concrete burster slabs, together with complete sandbag protection, would have to be installed just for the purpose of affording a minimum degree of protection. The cost of such protection not only would be enormous, but security would be doubtful.

Under the circumstances, the writer recommended the use of an auxiliary supply through driven wells. Because of conditions peculiar to the city being studied, this arrangement is much more economical and practical, and will offer a maximum assurance that a reasonable quantity of safe water will be available for domestic use and fire-fighting purposes. The question of whether to construct the auxiliary supplies at once will depend more or less upon the possibility of enemy bombers discharging their bomb loads over a given locality.

The conclusions reached, including the recommendations submitted, are based upon an analysis of the water works system, fire equipment, sector population densities, possible conflagration hazards, the geographical location and topography of the community. The auxiliary water supply has also been considered from the standpoint of engineering soundness, economy, practicability, and the ultimate effect upon public morale.

MICHAEL J. BLEW,[12] ESQ.[12a]—The greatest danger from sabotage of water systems is the interruptions to service caused by physical damage wilfully perpetrated against water works structures, of which intakes, dams, storage tanks, pumping equipment, and valves are generally the most vulnerable. As a rule, the treatment plant is more concentrated and better lighted, with workmen constantly moving around, rendering sabotage more difficult. Other water works structures are generally more isolated and therefore more exposed to the malicious tampering of saboteurs. In general, protection against sabotage consists of all the ordinary measures undertaken to prevent any interruption to service by a well-organized and well-operated water works department, plus added vigilance and added safeguards.

The development of additional or auxiliary portable laboratory equipment might be desirable to augment existing facilities or to provide temporary facilities when made necessary because of damage to the laboratory building by sabotage or bombing.

Much has been written concerning sabotage with chemical agents and pathogenic bacteria. The laboratory should be able to pick up most such attempts at sabotage without great difficulty. Again, vigilance is the predominant factor. Laboratory tests should be conducted more frequently and many more observations should be taken. Only a few additional tests ordinarily will need to be added. The problem is more serious for small water works than for larger ones.

Any chemical agent introduced into a water system with an intent to interfere with normal operation must be highly toxic in very low concentrations. Large quantities must be used to be effective. Such large quantities necessarily must be used that it would be difficult to avoid detection in the act of introducing them to the system. To be effective from a saboteur's viewpoint, such chemical agents must be cheap, easily obtainable, and difficult of detection (colorless, odorless, and tasteless). Not many toxic chemicals are available having these qualifications. The introduction of practically any easily available toxic chemical would result in an increase in turbidity (a haze), an alteration in pH, color production, or an increase in the chloride, sulfate, or nitrate radicals. All of these results are easily detectable.

The use of bacteria is always a possibility, but their use on a large scale is thought to be doubtful. Any bacteria, to be suitable from the standpoint of a saboteur, must be highly pathogenic, easily cultured, viable in water at a temperature lower than body temperature, and resistant to rapid changes in temperature, dilution, and dispersion. Special laboratory facilities must, of necessity, be available for the culturing of pathogenic bacteria in very large quantities. Such equipment is not easily available to unauthorized or unaccountable persons.

The concentration of bacteria thus introduced would, of necessity, be small, the concentration being dependent on the size of the water system into which the bacteria were introduced. The constant presence of a chlorine residual in the treated water would be almost practical proof of the nonexistence of any

[12] Lt. Col., San. Corps, Surgeon's Office, 4th Corps Area Headquarters, Atlanta, Ga.

[12a] Received by the Secretary April 30, 1942.

such bacteria in the supply. Pathogenic organisms, as a rule, are much more easily destroyed by free chlorine than non-pathogens. The only exception is in the case of spore-forming pathogens, which are much more resistant. In this case, the saboteur is limited to three or four species of bacteria, all of which are anaerobes and most difficult to culture in large quantities without special equipment and special care.

It is thought that all municipal and military water supplies should maintain a constant residual of free chlorine throughout the system. When there is any doubt, the residual should be increased. Many municipal supplies do not provide a chlorine residual. Frequent tests for free chlorine residual should be made at several points in the system, and a record should be kept as a control. The provision of facilities for the temporary chlorination of supplies in the event of a break in permanent equipment is most desirable. An ample supply of chlorine or chlorine compounds should be maintained at all times.

The admission of either chemical agents or bacteria to the raw water supply ahead of treatment would be much less effective than admission to the finished water in the clear-water well or distribution system. Most of these agents, especially bacteria, would be removed in the course of treatment by natural processes of oxidation and sedimentation. The contamination of a running stream is extremely doubtful. Contamination of an artificial reservoir, impounding lake, or natural lake would be easier, but treatment would very likely minimize it or render it inactive. Many toxic chemicals hydrolyze in water, forming new chemical compounds. These new compounds are sometimes toxic themselves and sometimes not. In some cases, the products of hydrolysis are insoluble or less soluble than the original chemical agent. This is particularly true of some of the warfare gases, especially those containing sulfur, arsenic, or phosphorus. Coagulation and filtration will remove these compounds to such an extent as to render them innocuous.

Admission of chemicals or bacteria to well-water supplies generally would be more difficult than for plants utilizing surface water. Danger of physical sabotage in all cases is far greater. The psychological factor in the case of chemical and bacterial sabotage is more dangerous than the actual results.

Reliance can be placed upon a well-trained, properly instructed operating force of faithful employees. Such a force is always a necessity to any water department, but it becomes much more so in wartime when danger of sabotage becomes greater. Where possible, reliance also can be placed on the fencing of strategic points; on watchmen and guards; and on electric wiring, electric eyes, automatic signals, etc.

Equipment should be placed in first-class operating condition. The employees should be examined and the untrustworthy ones "weeded out." Replacements of personnel should be made from the ranks of those found to be reliable. A repair and maintenance crew should be developed and maintained. They should be properly instructed as to their duties in the case of an emergency, and should be easily available when their services become necessary.

Danger from sabotage of sewerage systems or sewage treatment plants is second in importance to sabotage of water systems. In the case of damage from bombardment, they are of equal importance. Interruption of sanitary

sewer service by physical damage from bombing or by sabotage produces three principal effects:

(*a*) Menace to health through the flooding of basements with sewage, sewage from broken sewers being forced under pressure into broken or damaged water mains, back siphonage, etc.

(*b*) Inconvenience and menace to health by the disruption of sanitary service in homes, hospitals, factories, etc.

(*c*) Physical damage to structures (sidewalks, streets, foundations, and gas, electric, telephone, and water service) by undermining, washing out, and collapse.

Portable pumping equipment should be available for clearing flooded basements, sidewalks, vaults, etc., of sewage. Flooded basements should be sterilized with calcium hypochlorite, washed thoroughly, and dried out. Food contaminated with sewage should be destroyed.

Intercepting sewers and sewage treatment plants may be damaged by sabotage, but there is generally too much activity around these plants and too much light. Isolated pumping stations should be protected by fences, guards, or automatic signal alarms. Facilities should be provided for by-passing the entire treatment plant with chlorination of sewage if necessary.

One feature of the sewerage system that frequently is overlooked is the unprotected outlets of large sewers into creeks and stream beds. These openings afford easy access back from the stream under factories, which frequently are of a defense nature. Such openings should be protected from the unauthorized access of saboteurs. On such sewers, locked manhole covers are desirable and should be installed in any instance where large sewers flow under a defense industry and discharge directly into a stream, thereby affording an easy means of ingress.

In conclusion, although there is a danger of sabotage from toxic chemicals or pathogenic bacteria, it is thought that the greatest danger of sabotage lies in the threat of physical damage to water systems. An intensified laboratory program will largely minimize the threat of bacterial or chemical sabotage. Watchfulness and vigilance will minimize that of physical sabotage.

The menace to health from a disrupted sewerage system should not be minimized. It is thought that saboteurs will be more likely to attempt to use a large sewer system as a means of sabotaging an industry or defense activity than to sabotage such a system itself.

GILBERT H. DUNSTAN,[13] ASSOC. M. AM. SOC. C. E.[13a]—This Report should be of great value to all those engaged in the various services discussed. Although the material presented is directed to those who are engaged in the various activities considered, it seems desirable to provide the means of conveying some information direct to the general public.

Discussing water supply, the Report states (see heading "11. Distribution Systems: 12. Water and Sanitation Facilities") that "Announcements should

[13] Associate Prof., San. Eng., Univ. of Alabama, University, Ala.

[13a] Received by the Secretary May 1, 1942.

be made, and signs should be available, warning people regarding the use of the water until it is safe, and giving instructions as to the location of sanitary facilities." The writer feels that this is very important, but that it scarcely goes far enough. Some kind of card, very briefly outlining the proper emergency procedure, could be given to each customer. The card should contain the suggestion that it be hung in the kitchen or other convenient place in the home, and that all members of the family be familiar with it. This idea has been adopted in some places, one example being that of the Portland (Me.) Water District.[14]

The Report suggests possible community-wide distribution of bactericidal compounds and solutions for the use of individuals, and it would appear that this is worthy of consideration. However, many homes normally have cleansing solutions which contain chlorine, and which can be used to disinfect water, following the instructions on the bottle. It might be well, if public attention is called to the use of such solutions, to give a warning concerning the danger of using weak or deteriorated solutions that have been standing around for some time. Since iodine is often found in homes for use in first aid, attention should be called to its use as a water disinfectant. Ten drops of the mild antiseptic (2%) solution, or three drops of the 7% tincture of iodine, may be mixed with a quart of water by stirring; then let the solution stand for a half hour. If a card is furnished each home, instructions regarding the use of such disinfectants should be included.

Plans for emergency comfort stations are mentioned, and might become of great importance if the water supply of any section of a city should be interrupted for more than a few hours. Some means of disposing of excreta would be required in such cases, unless emergency comfort stations could be used in place of the home facilities normally utilized.

[14] "A Water Supply Defense Program," by J. Arthur Jackson, *Water Works and Sewerage*, Vol. 89, No. 3, March, 1942, pp. 107–109.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

HYDRAULIC DESIGN OF DROP STRUCTURES FOR GULLY CONTROL

Discussion

BY WALTER T. WILSON, ASSOC. M. AM. SOC. C. E.

WALTER T. WILSON,[16] ASSOC. M. AM. SOC. C. E.[16a]—During the years 1934 through 1938 the writer designed, supervised construction of, and made field inspections of, many gully-control structures. Literally hundreds of reinforced concrete and masonry erosion-control structures came to the writer's attention during this period. Had the research described in this paper been available at that time, great savings would have resulted not only from the economy of rational design, but also from the reduction of losses from under-design and subsequent failure.

Although the authors stressed the fact that they recognized that hydraulic limits are only part of the problem, it is relevant to mention a few of the others. The length of the floor must be great enough to provide adequate fill slope, top width of fill, and reasonable freeboard. The floor of the structure must be long enough to prevent overturning or "floating" of the structure from the hydrostatic head of the impounded water retained by it.

Lateral scour seems to have been given too little attention. Toe walls and downstream wing walls or retaining walls have failed from the erosion of part of the fill or of the original gully banks by the water discharging from the structure and forming a vortex. Attempts to control this lateral scour have included pervious pile wing dams radiating out from the sides of the structure.

The study of seepage past the structure may be included logically under the subject of hydraulics. The dimensions of the cutoff walls and other parts of the structure must be considered in the light of Darcy's law relating the static head to the path of flow or the length of seep or creep lines.

In the upper Mississippi region it was found that under many conditions the economical type of gully drop structure was either a drop inlet or a flume. Drop inlets, consisting of a box culvert with riser, have many advantages over

NOTE.—This paper by B. T. Morris, Jun. Am. Soc. C. E., and D. C. Johnson, Assoc. M. Am. Soc. C. E., was published in January, 1942, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1942, by John Hedberg, Assoc. M. Am. Soc. C. E.; and May, 1942, by Messrs. L. Standish Hall, and J. E. Christiansen.

[16] Asst. Hydrologic Engr., Weather Bureau, Washington, D. C.

[16a] Received by the Secretary April 16, 1942.

the open-notch type of structure. One of the advantages is the provision for a roadway across the gully at the site. An early laboratory study of the hydraulics of gully-control structures was reported at the University of Wisconsin in 1934.[17]

The writer believes that the most effective energy dissipater is a fairly deep stilling pool. He has seen many structures with a horizontal floor that were either too high or too low relative to the gully floor. In one case the floor might be silted full, eliminating the effectiveness of an acrobatic outlet as an energy dissipater. In the other case the floor might be left hanging, with danger of undercutting.

A properly designed flume with the lower end carried far enough below the subgrade has many advantages. An adequate stilling pool forms naturally. Variations in the elevation and gradient of the gully bottom from season to season can be accommodated. The flume, consisting of a hypotenuse in contrast to the vertical and horizontal elements of a vertical drop structure, naturally requires less materials.

[17] "Erosion Control Structures—Drop Inlets and Spillways," *Research Bulletin No. 122*, Univ. of Wisconsin, Madison, Wis., June, 1934.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

ANALYTICAL AND EXPERIMENTAL METHODS IN ENGINEERING SEISMOLOGY

Discussion

BY MESSRS. GEORGE W. HOUSNER, AND HOMER M. HADLEY

GEORGE W. HOUSNER,[42] JUN. AM. SOC. C. E.[42a]—In so far as it treats the earthquake problem from the viewpoint of vibration theory, this paper should prove of interest, particularly to engineers working in regions of seismic activity. That the effect of an earthquake is dependent on the physical properties of structures has long been known, and it was early recognized that the period of vibration of a structure was a significant index of its properties. One of the first attempts to measure the intensity of strong-motion earthquakes was made by Robert Mallet (1810–1881), a famous British engineer who made important contributions to seismology. The method used by Mallet[43] was to set up a group of six cylindrical wood blocks of graduated heights. By observing which of the blocks were overturned, he hoped to be able to classify the destructiveness of the earthquake. This wood-block approach has continued to interest engineers to the present time, but the rocking motion of a block is so complex that it has not been possible to draw any conclusions from the behavior of such a set of blocks.

A more satisfactory approach was taken by the late K. Suyehiro (1877–1932), noted Japanese engineer and one-time head of the Earthquake Research Institute of Japan. Suyehiro constructed a seismic vibration analyzer[44] to measure the intensity of earthquakes for different periods. This instrument[45] consisted of thirteen oscillators of different periods of vibration ranging from 0.22 to 1.81 sec. Such an instrument determines thirteen points on an earthquake spectrum such as that presented by the author. A defect of the instrument was the insufficient number of oscillators—that is, the gaps between the

NOTE.—This paper by M. A. Biot, Esq., was published in January, 1942, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1942, by Messrs. George R. Rich, N. J. Hoff, and Merit P. White; and May, 1942, by Messrs. N. H. Heck, Frank Neumann, and Jacob Feld.

[42] U. S. Engr. Office, Los Angeles, Calif.

[42a] Received by the Secretary May 5, 1942.

[43] "Dynamics of a System of Rigid Bodies," by E. J. Routh, Macmillan Co., 1913, p. 175.

[44] "A Seismic Vibration Analyzer," by K. Suyehiro, *Proceedings*, Imperial Academy II, 1926 (Tokyo), p. 286.

[45] A brief description of the instrument may be found in "Engineering Seismology: Notes on American Lectures," by K. Suyehiro, *Proceedings*, Am. Soc. C. E., May, 1932, Pt. 2.

periods recorded were too large. Acting on a suggestion from Professor Martel, the San Francisco office of the U. S. Coast and Geodetic Survey constructed an instrument on this principle with a larger number of oscillators whose maximum displacement is recorded.

The strong-motion earthquake records furnished by the U. S. Coast and Geodetic Survey since 1932 have made it possible to compute the response of oscillators of different periods, thus greatly increasing the scope of this phase of seismological research. This computation can be done in several ways—namely, by numerical integration as was done by M. P. White[6] at the California Institute of Technology in Pasadena, and by F. Neumann[46] of the U. S. Coast and Geodetic Survey; by actuating a shaking table and measuring the response of oscillators as was done by Professor Jacobsen[7] at Stanford University in Stanford University, Calif.; or by utilizing the torsion pendulum as was done by Mr. Savage[9] at the Bureau of Reclamation and also by Professor Biot..

A defect in this approach, as pointed out by the author, is that the computations are based on records of unknown accuracy. (A testing program to determine the accuracy of these strong-motion seismographs is being conducted at the Massachusetts Institute of Technology, in Cambridge, for the U. S. Coast and Geodetic Survey. The results of this investigation have not yet [June, 1942] been made public.) Any inaccuracy in the accelerogram will be reflected in the computed oscillator response curve or spectrum. It would be well for the author to call attention to the fact that the most reliable point on the computed spectrum is that for a period equal to zero. As stated by Professor Biot, the value of the computed oscillator response curve or spectrum is much increased by virtue of its application to structures more complex than a simple vibrating mass. This is particularly true since the vibration theory required has already been developed in the theory of acoustics. It is interesting to note that a building with shearing deformations is the mathematical analogue of a vibrating string, and the building with flexural deformations is the analogue of a tuning fork. This, then, is another instance in which the mathematics required by engineers has already been developed by physicists in the solution of analogous problems; even the analogue of the building with the flexible first story has been treated by them.

Since 1936, a program of earthquake research, sponsored by the Los Angeles County (California) Department of Building and Safety, has been conducted at the California Institute of Technology under the direction of Professor Martel. During this time spectra or oscillator response curves have been evaluated for the following earthquake records: Los Angeles Subway Terminal, March, 1933; Vernon, Calif., March, 1933; Los Angeles Subway Terminal, October, 1933; El Centro, Calif., December, 1934; Helena, Mont., October,

[6] "Some Studies on Earthquakes and Their Effects on Constructions," by R. R. Martel and M. P. White, Rept. on Earthquake Studies for Los Angeles County, Pt. I (1939) (unpublished).

[46] Mr. Neumann also called attention to the use of the torsion pendulum for this problem in "Special Publication 201, Earthquake Investigations in California 1934–1935," Dept. of Commerce publication.

[7] "Some Studies on Earthquakes and Their Effects on Constructions," by R. R. Martel and M. P. White, Rept. on Earthquake Studies for Los Angeles County, Pt. II (1939) (unpublished).

[9] "Earthquake Studies for Pit River Bridge," by J. L. Savage, *Civil Engineering*, August, 1939, pp. 470–472.

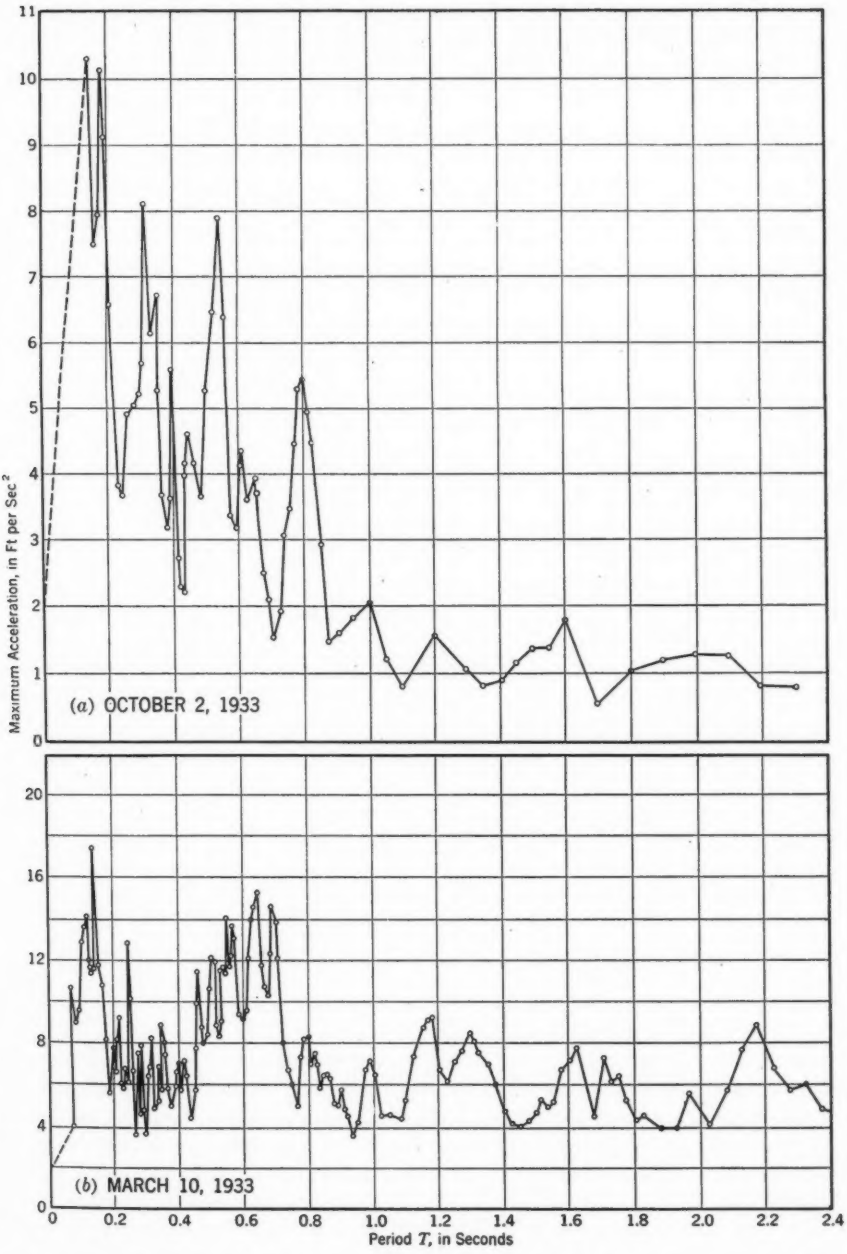


FIG. 11.—SPECTRA OF THE N 141° W COMPONENT OF THE LOS ANGELES SUBWAY TERMINAL RECORD

1935; and El Centro, May, 1940. In engineering terminology, these so-called spectra are in reality influence lines for maximum shear. The influence lines or spectra for the aforementioned earthquakes were found to be of two general types. These are illustrated in Fig. 11 where it will be noted that the records used were obtained from the same seismograph but for different earthquakes. The curve in Fig. 11(*a*) is similar to those presented by the author—that is, the ordinates decrease rapidly for increasing periods of vibration. In Fig. 11(*b*) this rapid decrease does not occur. It is seen that in this case the shear is not reduced, in general, appreciably by lengthening the period of vibration.

The problem of foundation yielding discussed by the author is a phenomenon well worth studying. It was first investigated by Suyehiro and Ishimoto[47] in 1926. These investigators constructed a special vibrograph and made measurements on the rocking oscillations of buildings.[45] Some results of their measurements are shown in Table 4. These were made on reinforced

TABLE 4.—OBSERVED PERIODS OF VIBRATION

Laboratory	Stories	Meters			Ground	Period (sec)
		Length	Breadth	Height		
Aeronautics; main building	2[a]	56	15	11	Reclaimed	0.50
Mitsubishi Laboratory; main building	2[b]	60	14	10	Loam	0.35
Mitsubishi Laboratory; metallurgical room	1[a]	11	9	5	Loam	0.20

[a] No basement. [b] With basement.

concrete buildings of very rigid construction. To compare values with the results of the author's analysis, it is necessary to recompute Table 3. In his calculations, Professor Biot has used the value $p_s = 84$ lb per sq in., apparently confusing allowable bearing pressure with actual building weight. On the basis of the dead load of typical buildings, it would be proper to take $p_s = 1.04$ lb per sq in. for each 12 ft of height. Recomputing on this basis gives the following values:

Values of r (ft)	For $2L = 50$ ft, period T, in seconds, equals:
16	0.04
50	0.35
100	0.99

Only the structure 50 ft wide has been computed and the value $r = 16$ represents a height of 12 ft. As recomputed, it is seen that the calculated periods of the rocking oscillations are much shorter than the corresponding periods of the first modes of vibration would be. These computations would indicate that foundation yielding does not cause an appreciable stress reduction due to period lengthening.

A very extensive program of vibration measurements is reported in "Special Publication 201."[46] The periods of 212 buildings were measured and, of these,

[47] "Vibrations of Buildings," by K. Suyehiro and M. Ishimoto, *Proceedings*, Third Pan-Pacific Science Cong., Tokyo, 1926, p. 1482.

four showed evidence of possible foundation yielding. These four were low, rigid buildings on soft ground. The building vibrations (due to microseisms) measured by Suyehiro and Ishimoto[47] would also indicate that foundation yielding may be of importance for low, rigid structures on soft ground. It is of interest to note that Suyehiro concluded that the rocking oscillations were nonlinear since the period was dependent on the amplitude. The approach used by the author would not be applicable to nonlinear vibrations, although in itself nonlinearity would seem to be beneficial in reducing maximum shears. It appears that more experimental research is required to determine the fundamental characteristics of rocking oscillations of buildings.

HOMER M. HADLEY,[48] ASSOC. M. AM. SOC. C. E.[48a]—The gist of Professor Biot's scientific approach to the problem of earthquake resistant construction and to the improvement of codes appears to lie in his advocacy that recognition be given to the varying responses which different structures having different periods of vibration give to the same earthquake and, further, that recognition be given to varying types of foundations with their differing elastic properties and consequent differing effects upon structures built upon them. That such differences exist cannot be questioned, and it would seem entirely reasonable and logical to include consideration of these matters in design. When specific application is attempted, however, the engineer finds himself embarrassingly deficient in reliable data upon which to proceed. A sage saying which the writer heard years ago in Alaska was this: "Now, if we only had some ham, we'd have some nice ham and eggs for breakfast—if we only had some eggs." In short, for ham and eggs one must have both ham and eggs. Even so, for the application of the scientific method, the engineer must have reliable data for its basis. What is definitely known about earthquakes, about foundations, about the structures themselves?

One scientific fact about an earthquake always faithfully reported is the location of the epicenter. Not much else may be learned of it, but the location of the point on the earth's surface above the subterranean origin is readily determined. From the accumulation of these data engineers have come to know what regions are most subject to earthquakes, but beyond this there is no particular application of the knowledge made to earthquake resistant construction. Even the position of the epicenter is subject to question at times. At Long Beach, Calif., on March 10, 1933, the epicenter was first reported in the ocean bed some 15 miles off Newport Beach, Calif., where only slight damage occurred.[49] It seemed somewhat anomalous that the severe damage should have occurred at places much farther removed from the epicenter than Newport Beach. Later observations[50] led to the conclusion that there had been a series of epicenters at various points along the Inglewood, Calif., fault, there being a series of successive origins. Since these later epicenters were

[48] Regional Structural Engr., Portland Cement Assn., Seattle, Wash.

[48a] Received by the Secretary May 13, 1942.

[49] "Preliminary Report on the Long Beach Earthquake," by Harry O. Wood, *Bulletin*, Seismological Soc. of America, April, 1933.

[50] "The Determination of the Extent of Faulting with Application to the Long Beach Earthquake," by Hugo Benioff, *ibid.*, April, 1938.

adjacent to the districts of severe damage, that much empirical confirmation of their existence is found.

As to the earthquakes themselves, the various records show certain common characteristics: An initial period of slight movement; then the period of severe, violent movement; and then the final period of diminishing, dying movement. As to their intensities, it is known that they vary widely. The accelerograms of the two Ferndale, Calif., earthquakes of February 6, 1937, and September 11, 1938,[2] are quite different. Who, on March 1, 1937, for instance, having seen the accelerogram and spectrum of the February 6, 1937, earthquake and of all preceding earthquakes at Ferndale, could have said that a more severe earthquake would occur on September 11, 1938, or, on any particular date, could have foretold what its accelerogram or spectrum would look like? Of necessity, such a prophet would have to employ "art based on observational facts and experience" rather than "science" as the basis of his prediction; so it is, with Ferndale, Calif., or with any Ferndale, large or small, upon this earth. Somewhere in his writings, Omori discussed the earthquake peculiarities of certain places and stated that there were individual characteristics and features to be observed; but except for a certain directional similarity there was nothing strikingly individualistic in these characteristics. Now, and probably for a long time in the future, engineers must be empirical in judging the earthquake to be assumed in design.

As to the influence of the foundations on the motion of the structure, Professor Biot clearly states the complexity of the problem (see heading "V.—Influence of Foundation on Motion of Blocks"): "* * * it involves a complete knowledge of the propagation and properties of the seismic waves in the strongly heterogeneous surface layers of the earth as well as their diffraction and reflection by objects built on the surface." This undoubtedly is true and undoubtedly presents an obstacle to "science." It is stated that Eq. 38 implies the knowledge of the elastic constants E and μ of the soil but that "These are obtainable from tests made by M. Ishimoto and K. Iida.[13]" This is a happy solution of the difficulty: See what Messrs. Ishimoto and Iida give for these values. Nevertheless, it is to be noted in their quoted findings that a change of moisture content of silty clay from 42% to 50%—that is, a change of 8%—changes E from 792 lb per sq in. to 5,750 lb per sq in. Who is to know whether what he terms silty clay is the same as the silty clay defined by Messrs. Ishimoto and Iida, or what the moisture content of that silty clay may be when a foundation is subjected to an earthquake? Again, what manner of bedrock underlies the silty clay? Those who have driven the highways of Southern California and looked upon the confused, contorted, heterogeneous geologic structure revealed in cuts and excavations must feel mistrust and misgivings in whatever assumptions they may make as to the values of E and μ for scientific calculations.

As to the elastic properties of the structure that is being designed, it should be possible from measurements, past and future, of existing comparable

[2] "A Mechanical Analyzer for the Prediction of Earthquake Stresses," by M. A. Biot, *Bulletin*, Seismological Soc. of America, Vol. 31, No. 2, April, 1941, pp. 151–171.

[13] "Determination of Elastic Constants of Soils by Means of Vibration Methods," by M. Ishimoto and K. Iida, *The Bulletin of the Earthquake Research Institute*, Vol. XIV (1936), Pt. I; also *ibid.*, Vol. XV (1937), Pt. II.

structures to assume values fairly approximate to those that will develop. With values of T decided on, with what spectrum shall they then be used? Apparently there is only a very limited number in existence. Fig. 6 is a "standard" spectrum of two earthquakes only—that of Helena, Mont., on October 31, 1935, and the Ferndale earthquake of September 11, 1938.[50a] It shows that a natural period of 0.5 sec gives an acceleration of 0.4 g; a natural period of 1.0 sec gives an acceleration of 0.2 g. From the text one learns (see heading "III.—Spectrum Related to Design and Actual Behavior of Structures") that "The differences [in the behavior of buildings of different periods] are not as great as the spectrums would indicate." Apparently "Science" must be empirically modified, at times.

There is a further matter of engineering seismology to be considered, as well as the matter of intensity of stresses, which can be modified and altered by change of sections. This is the matter of greater deformations that accompany longer periods. The Maranouchi Building in Tokyo, Japan, the largest steel-frame building in that city, was under construction in 1922 when a considerable earthquake occurred. As a result of the distortions it sustained at that time, it was felt that the building needed stiffening. Consequently, a number of panels of diagonal bracing were introduced, extending from basement to roof, and these were covered with metal lath and plaster. When the great earthquake of September 1, 1933, occurred, these bracing panels were loaded terrifically at the outset and either stretched amazingly or ruptured. The natural period of the building then reverted to its original value. A number of different people who were in the building told the writer that they did not enjoy the experience. Partitions were cracked and shattered, marble trim in the corridors loosened and fell, and pendant lighting fixtures swung so violently that their reflectors knocked plaster off the ceilings. After it was all over the owners—perhaps not scientifically, yet empirically—rebuilt the exterior and interior walls and introduced bracing walls, for the definite purpose of making the building stiff—that is, giving it a low natural period of vibration. States a well-known advertisement: "Ask the man who owns one."

It must not be thought that making a building stiff will prevent an earthquake from being felt in it. On the contrary, whatever movement occurs at the foundations will be transmitted with little damping to the roof, and the building's contents will be shifted and perhaps thrown to the floor quite as if they stood upon the ground. On the other hand, after the earthquake has passed, a well-designed, rigid-type building will be found very satisfactory to all concerned. It may be added that all the comment made herein is limited to buildings of moderate height. As to skyscrapers, the writer has no knowledge other than that such buildings successfully passed through the San Francisco, Calif., earthquake of 1906.

Yet another aspect of the earthquake problem is the matter of occurrence. Fortunately, severe earthquakes are not frequent. It may well be questioned whether provision against them warrants highly elaborate and refined engineering design. The severe damages which have repeatedly occurred in the past have not resulted from failure to design for this or that

[50a] See corrections to Fig. 6 in April, 1942, *Proceedings*, p. 611.

value of shearing stresses in the structures involved but from completely ignoring earthquakes and all horizontal forces they entail. A definite observed deficiency in present design or proof that current practise is unwarrantedly too severe should constitute the basis of change and progress.

There is a widespread feeling among American engineers that some special virtue is imparted to the solution of highly complex, not to mention uncertain and unknowable, problems by going through mathematical processes in reaching a decision regarding them. They prefer to assume E and μ and all the various other variables that may be involved and to "figure" themselves to an answer rather than to accumulate empirical data and proceed simply and directly from them. There is much to be said for the mathematical process, but it contributes no more truth, no more science, than is put into its basic assumptions. The writer would conclude with the words of the late David A. Molitor, M. Am. Soc. C. E., used in the "Synopsis "of his paper "Wave Pressures on Sea-Walls and Breakwaters":[51] "Since all factors contributing to the solution of the problems herein considered are only approximately knowable, no theoretical approach was deemed advisable."

[51] *Transactions*, Am. Soc. C. E., Vol. 100 (1935), p. 984.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

TECHNIQUE OF DETERMINING SHEARING STRENGTH OF SOILS

PROGRESS REPORT OF A SPECIAL COMMITTEE OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION ON THE TECHNIQUE OF SOIL TESTS

Discussion

BY JACOB FELD, M. AM. SOC. C. E.

JACOB FELD,[3] M. AM. SOC. C. E.[3a]—This one-man committee report by Mr. Hough is a complete and excellent presentation of the subject. As stated, there are really three questions involved:

(1) What type of apparatus shall be used for testing?
(2) What methods should one use to get the results of the test?
(3) What should one do with the results after obtaining them?

After all, the test is not an end in itself, but a means of evaluating, numerically, the expected action of the soil in or adjacent to an engineering structure.

It differs little to the engineer what type of apparatus or what procedure is used to obtain results, if the results obtained can be applied. That means that if the results obtained are applied to the design of a structure, the structure will act in the way expected and in the way indicated by the results of the tests.

Generally, any test used as a guide for the correlation of field data obtained in actual practice on previous work with a new set of conditions which arises and which is within, or possibly just outside of, the conditions encountered in previous work will be found useful and applicable. On the other hand, the use of any test results obtained from a 1-in. cylinder of soil, or even a cylinder of somewhat larger size, by extrapolation, for designing an earth dam or even a footing of a one-story building, where no information is available as to the

NOTE.—This Report was published in February, 1942, *Proceedings*. Discussion on this Report has appeared in *Proceedings*, as follows: February, 1942, by Glennon Gilboy, Assoc. M. Am. Soc. C. E.; and April, 1942, by Cloyd D. Beerup, Assoc. M. Am. Soc. C. E.

[3] Cons. Engr., New York, N. Y.

[3a] Received by the Secretary April 29, 1942.

action of similar structures on similar soils, will prove to be dangerous and unsatisfactory.

A practical example of the meaning of the foregoing statements may clarify the picture: The resistance to driving a shield in subaqueous tunnels at two locations was known, and samples of the mud or silt through which each of the tunnels had been driven were available. These samples were far from undisturbed; actually they were merely milk bottles full of the mud obtained during the construction of each tunnel. Although they were sealed with paraffin, they fell far short of supplying a perfect picture of the true nature of the material encountered. Some information was desired as to the expected resistance to driving a tunnel in a third location, and some of the material taken in soundings in that location was available. This material, again, was not in an undisturbed condition; nor was it a true picture of the physical characteristics of the material to be encountered. The three samples were taken to a laboratory and, in a saturated state, a simple shear test on various loadings, similar to the direct-shear friction test shown in Fig. 1(*a*), was performed. The results of the tests of the samples obtained from the completed tunnels gave two points on a graph, plotted as a frictional coefficient against resistances per unit of the circumference of the tunnel. The frictional test on the third sample indicated a corresponding resistance for the tunnel. The actual value as determined during the construction of the third tunnel checked the expected value quite closely.

Unless the technician knew the answer in the two previous cases, no amount of laboratory technique in testing the third sample could have given the expected information with any degree of accuracy. The application of the foregoing laboratory soil tests is of great benefit. The attempt to apply soil tests without correlated field experience and knowledge is a dangerous procedure and has resulted in some non-complimentary reports on the application of soil mechanics to engineering design.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

VISCOSITY AND SURFACE TENSION EFFECTS ON V-NOTCH WEIR COEFFICIENTS

Discussion

BY A. A. KALINSKE, ASSOC. M. AM. SOC. C. E.

A. A. KALINSKE,[6] ASSOC. M. AM. SOC. C. E.[6a]—Any engineer concerned with the measurement of discharge of various liquids by the use of the V-notch weir should welcome this most noteworthy addition of experimental data. The precision of the author's measurements and the wide variation of viscosity and angle of notch are the outstanding features of this paper. The writer is particularly interested in the analyses of the data and therefore will confine this discussion to that phase.

The writer cannot agree that a general equation (Eq. 3) has been derived relating all the pertinent variables. The equation holds only for a certain range of Reynolds' and Weber's numbers since it was derived only for a range of data where the quantity $(C - 0.56)$ versus **R** and **W** plots as a straight line on log-paper. There is, of course, no physical reason why this should be true for all ranges of **R** and **W**. The situation is comparable to the pipe friction factor, f, versus **R** plotting for pipe friction for which various power functions can represent the relation between f and **R** for different ranges of **R**-values. It should be kept in mind that an empirical equation derived by fitting a curve to a certain range of data can scarcely be expected to be general, or physically sound. The fact that the equation is dimensionally consistent does not necessarily mean that it is generally applicable.

So far as the writer can determine, Eq. 3 was derived by the analysis of the data in Fig. 8. By themselves these data do not really prove that C is a function of **R** and **W** as indicated, since the viscosity and surface tension were not varied—only the head. The writer does not wish to imply that C may not be a function of **R** and **W**; he wishes only to state that viscosity and surface tension should be independently varied to demonstrate experimentally that **R** and **W** affect C in the manner indicated. The author has other data, of course, which prove that C is definitely a function of **R** and that it is related

NOTE.—This paper by Arno T. Lenz, Assoc. M. Am. Soc. C. E., was published in March, 1942, *Proceedings*.

[6] Asst. Prof. and Research Engr., Univ. of Iowa, Iowa Inst. for Hydr. Research, Iowa City, Iowa.

[6a] Received by the Secretary May 5, 1942.

to **W**, although the surface tension was not really varied over a very great range. However, it should be re-emphasized that the data in Fig. 8 are not in themselves sufficient to warrant the extensive use to which they are put.

Eqs. 11 and 12, which are derived from Fig. 8, are not in any sense different equations. Since **R** varies as $H^{1.5}$ and **W** as H^2, and since τ and σ were constant for the data in Figs. 8, the data plotted in these curves are actually a plotting of $(C - 0.56)$ against $H^{1.5}$ and H^2. Naturally, then, the exponent of **R** in Eq. 11 and that of **W** in Eq. 12 should be as the ratio $2.0/1.5 = 1.33$, which is approximately what the author determines, since $0.36/0.28 = 1.29$. The value of B_2 in Eq. 12 could be determined from B_1 in Eq. 11 and the expressions for **R** and **W**, since all terms in these expressions were constant except H. In other words, Eqs. 11 and 12 are not actually independent equations.

In the discussion of the limiting conditions in the use of Eq. 3 (see heading "Analysis of Data: Limiting Conditions for Eq. 3"), the author gives an expression (Eq. 15) for the critical value of **R**. He then states that the change in flow at this critical value is due to change in viscosity and not in surface tension or head. This is not quite true, since **R** is affected both by a change in viscosity and in head. In other words, the change in flow is due to a change in **R**.

In Section (*c*) of the discussion on the limiting conditions of Eq. 3, the author states that for infinitely large values of **R** and **W** the coefficient C becomes 0.56; however, he states further that no data bear this out. This is very easily explained: Eq. 3 is applicable for a limited range of **R**-values and **W**-values and the fact that it does not hold for all values is no more surprising than the fact that the Blasius relation for f and **R** for pipe friction holds only for a limited range of **R**-values.

The foregoing remarks do not, of course, invalidate the use of Eq. 3, since the author is careful to point out the limits of its applicability. The reasons for the limitations are quite obvious since a simplified empirical equation cannot be expected to describe completely a complex flow phenomenon when viscosity and surface tension are both influencing factors, as is the case in this problem. Although entirely empirical equations with clearly defined limitations are useful, a curve or curves indicating the values of C for all practical ranges of **R** and **W** probably would be the best so far as engineering use is concerned.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

PROFILE CURVES FOR OPEN-CHANNEL FLOW

Discussion

BY CARL ROHWER, ASSOC. M. AM. SOC. C. E.

CARL ROHWER,[5] ASSOC. M. AM. SOC. C. E.[5a]—There is considerable difference between the shape of certain types of profile curves as generally shown in textbooks and the shape obtained by the solution of the backwater formula. This is demonstrated clearly by Mr. Gunder. The question immediately arises as to which of these is the true shape of the profile in a stream or canal under the conditions producing this type of backwater curve. Obviously, both cannot be correct. If there is a point of inflection in the curves, as stated by the author, then the curvature will change from concave upward to concave downward and the length of the surface profile would be materially reduced, in consequence. If this change in the shape of the profile actually occurs in nature, it is strange that this fact has not been revealed before. Such a change scarcely could have passed unnoticed by engineers taking levels on profiles under the conditions necessary to produce this phenomenon. It seems more plausible to the writer that this discrepancy is caused by some error in the assumptions involved in the derivation of the backwater equations rather than in the failure to observe the physical facts. The discovery of this anomaly in the general equation of surface profiles leads one to wonder if the application of the equation to the solution of the shapes of other types of profile curves may not result in errors that are not so readily detected.

Although the study of surface profiles in open channels for gradually varied flow is of special interest to engineers concerned with the problem of determining the rise in the water level in streams above dams and locks, it is of importance, also, to engineers engaged in the design of irrigation structures. The writer's experience is limited to the special backwater curve produced when a Parshall flume[6] is installed in a canal. Since the Parshall flume constricts the channel, there is an increase in the depth of the canal above the flume. The amount of this increase has been determined by tests on flumes of various sizes for a wide range of discharges. This increase in depth extends upstream in the canal and,

NOTE.—This paper by Dwight F. Gunder, Esq., was published in April, 1942, *Proceedings*.

[5] Irrig. Engr., Div. of Irrig., SCS, U. S. Dept. of Agriculture, Colorado Experiment Station, Fort Collins, Colo.

[5a] Received by the Secretary May 22, 1942.

[6] "The Parshall Measuring Flume," by R. L. Parshall, *Bulletin No. 423*, Colorado Experiment Station, 1936.

since the head gate and diversion dam are usually only a short distance from the Parshall flume, the problem is to determine whether the rise in the canal will be sufficient to make it necessary to increase the height of the diversion dam.

Canal cross sections are relatively narrow and deep; consequently, it cannot be assumed that the hydraulic radius is equivalent to the depth. To determine the effect of backwater, it is therefore necessary to divide the canal into sections for which the length, corresponding to an arbitrarily chosen change in depth, can be computed. This is a tedious process. It is hoped that the author will derive a simple equation, applicable to the surface profile produced in this special case.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

NUMERICAL PROCEDURE FOR COMPUTING DEFLECTIONS, MOMENTS, AND BUCKLING LOADS

Discussion

BY M. S. KETCHUM, JR., ASSOC. M. AM. SOC. C. E.

M. S. KETCHUM, JR.,[20] ASSOC. M. AM. SOC. C. E.[20a]—The author's paper is another contribution to a distinctly American tradition of structural analysis founded on the work of Hardy Cross. This tradition is characterized by a non-mathematical solution of quite difficult cases in such a manner that the structural behavior is always evident in quantitative terms.

The writer would like to suggest a simple and convenient approximation that he has found to be useful in a large variety of buckling problems, including the lateral buckling of crane runway girders. The total deflection, w_0, in the strut analyzed in Fig. 17 may be estimated as soon as the deflection, w_i, due to the lateral load, is calculated and before the value of the critical load, P_{cr}, is determined. The assumption may be made that the shape of the bending-moment diagram for the moments, $P\,w_0$, due to the column load, is of the same shape as the bending moments due to lateral loads.

If M_L is represented as the moment due to the lateral load and w_i as the deflection due to this load, then the deflection, w_a, due to a moment, $P\,w_0 = P\,(w_i + w_a)$, is:

$$w_a = \frac{w_i\,P\,(w_i + w_a)}{M_L} \qquad (16)$$

Solving for w_0 in terms of w_i:

$$w_0 = \frac{w_i}{1 - \dfrac{w_i\,P}{M_L}} \qquad (17)$$

NOTE.—This paper by N. M. Newmark, Assoc. M. Am. Soc. C. E., was published in May, 1942, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: May, 1942, by Bruce Johnston, Assoc. M. Am. Soc. C. E.

[20] Asst. Prof., Structural Eng., Case School of Applied Science, Cleveland, Ohio.

[20a] Received by the Secretary May 21, 1942.

A substitution of the values of w_i, P, and M_L at the center of the strut gives:

$$\frac{w_i P}{M_L} = \frac{1{,}042 \times 0.02}{100} = 0.2084; \text{ and}$$

$$w_0 = \frac{1{,}042\, q\, \lambda^4}{8\, E\, I}\left(\frac{1}{1 - 0.2084}\right) = \frac{1{,}320\, q\, \lambda^4}{8\, E\, I} \quad \ldots\ldots\ldots (18)$$

The corresponding value from Fig. 17 has a coefficient of 1,306 so that the approximation is satisfactory for most problems.

The critical buckling load for this strut determined in Fig. 12 also may be estimated by the condition that $w_i = w_0$, and

$$w_0 = \frac{w_i P\, w_0}{M_L} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (19a)$$

and

$$P_{cr} = \frac{M_L}{w_i} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19b)$$

Substitution of values from Fig. 17 gives:

$$P_{cr} = \frac{100\, E\, I}{1{,}042\, \lambda^2} = \frac{9.60\, E\, I}{L^2} \quad \ldots\ldots\ldots\ldots\ldots (20)$$

The corresponding coefficient from Fig. 12 is 9.87 so that the approximation is in fair agreement.

The author is to be commended for his simple yet thorough and rigorous treatment of deflection and buckling. This paper should dispel much of the mystery that surrounds all but the most simple column problems.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

MODEL TESTS ON STRUCTURES FOR HYDROELECTRIC DEVELOPMENTS

Discussion

BY ADOLF A. MEYER, M. AM. SOC. C. E.

ADOLF A. MEYER,[6] M. AM. SOC. C. E.[6a]—In Mr. Davis' interesting paper is a description of the hydraulic laboratory tests which led to the adoption of the type of apron used at the Safe Harbor spillway and to the decision to install a similar one at the Holtwood Dam. Surprisingly, this type has received very little attention in engineering journals. This is unfortunate because, where conditions make it applicable, it offers the most economical means for safe energy dissipation below spillways.

This form of apron creates two typical conditions: It dissipates the energy in the natural river bed below the apron, and it separates the zone of scouring from the structure by lifting the body of the shooting water at the end of the upturned apron bucket.

The change from shooting to streaming flow is obtained by hydraulic jump in the natural river bed. When free surface rollers only occur, the scour is kept to a minimum. However, no analytical method allows prediction of the type of rollers for all discharge and tailwater conditions. When ground rollers occur, the scour may be very large. Only hydraulic model tests can give positive assurance about the behavior of the discharging water in the scouring zone.

There are a few specific conditions which prevent the use of this economical apron on certain dams. One is the character of the foundation rock. Obviously it would not be advisable to use this type where the foundation rock is bedded in horizontal layers with weaknesses in the bedding joints. Where expensive shore protection is necessary along the full length of the scouring zone, the over-all economy of this type may become questionable. As high wave action results from this type of energy dissipation, it will be objectionable where navigation channels come close to the downstream end of the dam.

NOTE.—This paper by L. M. Davis, Assoc. M. Am. Soc. C. E., was published in April, 1942, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1942, by Messrs. Emil P. Schuleen, and C. M. Allen.

[6] Head Civ. Design Engr., TVA, Knoxville, Tenn.

[6a] Received by the Secretary May 29, 1942.

During its ten years of operation at Safe Harbor, floods of unusual magnitude have passed the apron. It would be very interesting if Mr. Davis could show the present-day scouring picture at the prototype and indicate how it compares with the laboratory predictions.

The simple straight-line design for the apron at Holtwood is interesting. It may be noted here that the condition at Hales Bar Dam, construction of which was started the same year as Holtwood (1905), is in many ways similar to the condition at Holtwood. No apron was provided during the initial construction and some objectionable scouring developed, which has led to the decision to install an apron in the near future. However, due to the fact that a navigation channel is adjacent to the downstream end of the dam, an apron will have to be provided which dissipates most of the energy immediately below the dam in order not to disturb navigation by excessive wave action.

The TVA adopted the jump-type apron recently for the Apalachia and Ocoee No. 3 projects. At Apalachia, the energy per linear foot of apron is approximately the same as at Safe Harbor. Laboratory tests showed an apron with a 20°-bucket angle to give unusually satisfactory service. In the model, the sand bed immediately downstream from the apron remained undisturbed, with the scour never exceeding 10 ft in depth anywhere downstream from the apron.

Mr. Davis' interesting paper gives a clear illustration of highly satisfactory results which can be obtained when the design for hydraulic structures is extensively coordinated with hydraulic laboratory model tests.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

TIMBER FRICTION PILE FOUNDATIONS

Discussion

BY JOHN W. SOLOMON, ESQ.

JOHN W. SOLOMON,[11] ESQ.[11a]—There are many interesting facts in this paper that certainly will be useful to the practicing foundation engineer. It is always worth while to inquire into the relationship between the computed failure load (bearing power) of a group of friction piles and the actual load required to produce failure. By referring to experiences with friction piles of reinforced concrete the writer proposes to demonstrate that the differences between failure loads of parallel sided and tapered piles, reported by Mr. Masters, can be explained easily. In addition, the differences in the solution proposed by Professor Mindlin[2] and the so-called Boussinesq formula can also be explained.

In the work that the writer has in mind numerous reinforced concrete friction piles were moulded in the soil (cast in place) by first sinking a casing, then setting steel bars, and finally pouring concrete. The concrete was compacted by compressed air, and the casings finally removed. Such friction piles were constructed in the dried bed of a former river, and along the banks of a flowing river. The pile lengths varied from 40 ft to 50 ft, and the diameters from 16 in. to 17 in. The soil was a mixture of very fine sand, silt, and clay, with sometimes a pocket or stratum of medium sand.

Tests to failure, at the electric power station of Comines, in the northern part of France, gave skin friction determinations of 350 to 420 lb per sq ft. In the vicinity of the Alps, piles were driven for the towers of a high-tension line from Bissorte to Montmelian. Where this line crossed the River Arc, the skin friction was found to be only 320 lb per sq ft, the piles being 50 ft long, and the soil a very fine material composed of incongruent elements (mud). Some piles were tested by applying a horizontal pull at a point two thirds the height of the tower superstructure. This cantilever pull imposed compression

NOTE.—This paper by Frank M. Masters, M. Am. Soc. C. E., was published in November, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: February, 1942, by E. H. Connor, M. Am. Soc. C. E.; April, 1942, by Messrs. Glenn B. Woodruff, Jacob Feld, G. G. Greulich, and G. S. Paxson; and May, 1942, by Raymond D. Mindlin, Assoc. M. Am. Soc. C. E.

[11] New York, N. Y.; formerly Engr. in Chg. of Roumanian Pavilion, New York World's Fair, 1939 and 1940.

[11a] Received by the Secretary May 8, 1942.

[2] "Force at a Point in the Interior of a Semi-Infinite Solid," by Raymond D. Mindlin, *Physics*, May, 1936.

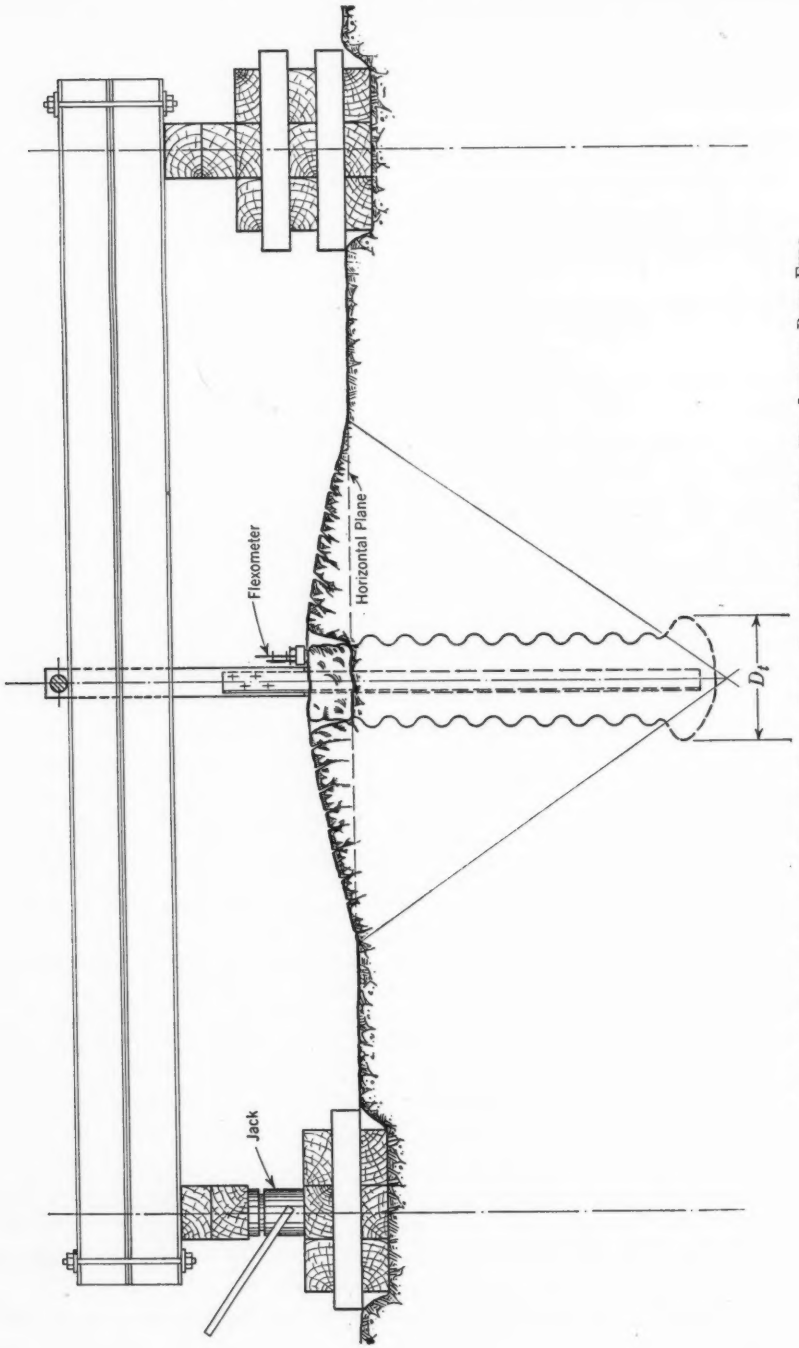


FIG. 13.—LOADING SETUP FOR PULLING TEST. FOR A PERFECT PULLING TEST USE JACK ON BOTH ENDS

on two piles, and tension on two piles. Deducting the point loads it was found that the piles in tension failed at smaller loads than those in compression. Similar tests with piles varying in length from 8 ft to 12 ft, of which 6 ft to 8 ft was embedded in pebbles and stones up to 6 in. diameter, developed a tensile resistance of from 60 to 65 tons. The compression of the concrete piles against the soil created some kind of a cementation of the soil. Conditions at the ground surface indicated that the area influenced by the test was a segment of an almost perfect sphere, with its center at the enlarged tip of the pile (see Fig. 13). For all practical purposes the simple theory of a tension cone was considered sufficiently accurate. For two piles adjacent to each other, the writer obtained the formula

$$K_R = \frac{l^3}{2\,(2\,L \tan \phi + D_t)^3} + \tfrac{1}{2} \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

in which K_R is a reduction coefficient; l is the distance between the centers of the two piles; L is the length of the pile; ϕ is the angle of the cone of failure; and D_t is the diameter of the enlarged tip of the pile.

In 1935 individual piles and groups of piles were placed for a large storehouse at Grande Couronne, near Rouen, on the River Seine. These piles were 70 ft long, driven into a solid chalk stratum. The soil above the chalk was a 25-ft to 30-ft stratum of fill and mud, overlying a stratum of sand and pebbles 6 ft or 8 ft deep, with alternating layers of fine sand and clay, 8 ft to 10 ft deep, to the level of the chalk. Twelve single piles were driven to a stand test load of 140 tons each (including the weight of the pile), and one group of nine piles, capped by a reinforced concrete slab, was tested at 1,350 tons. In all cases the load was applied by filling a box with iron pyrites. For the individual piles, in addition, a hydraulic jack was applied, and the settlement of the head of the pile was registered by two flexometers (see Fig. 14). The load was increased gently, at regular intervals, by increments of 5 tons. Precautions were taken so that the points of contact at the bottom of the loaded box were some distance from the piles, and the same precautions were made in fixing the support for the flexometers.

In the case of the pile group, the box of the pyrites was centered on the reinforced concrete caps. A load was applied by adding a constant quantity of pyrites to the box until the total of 1,350 tons had been reached. In this case, five registering flexometers, one at each corner, and one at the center, were used to record the settlement of the group. Every test proved that the load was transmitted to the soil from the pile, beginning quite close to the surface, and extending down to the tips. This was interpreted to mean that, for relatively small loads, only the friction surface of the upper part of the piles was working. The flexometer indicated an elastic contraction of the pile which was much less than could be expected under the assumption that the load, however great or small, was transmitted by the full length of the pile, regardless of the distribution formula adopted. At Grande Couronne the tip of the 70-ft pile began to "work" only after the superposed load had exceeded 50 tons. In any foundation project involving the use of piles, the test diagram of each pile and a record of the nature of the soil for the full depth of the pile are essential.

In the case of friction piles, an exact picture of the distribution of the load resistance at points adjacent to the entire depth of the pile require loading diagrams with load increments increased slowly. Cylindrical piles, driven in almost homogeneous soils, have a constant stress distribution, beginning at a certain depth below the surface. Diagrams of the tests, such as these, permit the investigator to study the building up of stresses in the soil, as a pile is loaded, and thus to determine the interaction between the piles of a given group. Especially it is possible to appraise the final distribution under the failure load.

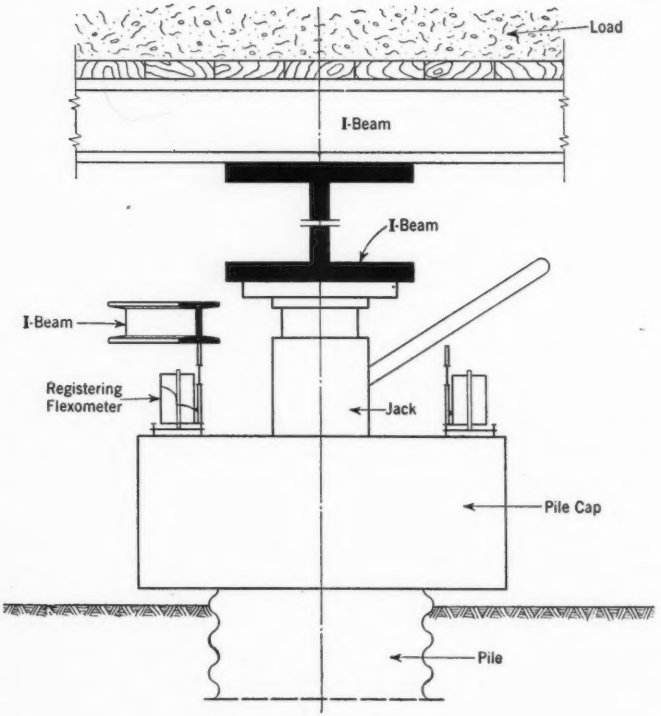


FIG. 14

The contraction of the pile itself has been neglected by Mr. Masters. Fig. 6, for example, showing shears for piles loaded to 10 tons, is not consistent with actual facts. Under only 10 tons per pile, no load would be transmitted at the tip. Eq. 2*c* was based on a positive value of the point load at the tip, greater than zero. There would be no load delivered by the tip for the piles in Fig. 6 when the applied load is 10 tons or less. The same applies to the 60-ft piles in Table 9.

Consider a friction pile in place, ready for its working load (see Fig. 15). If a load P_1 is applied, the upper length of the pile, L_1, shortens by an amount Δ_1, and the surrounding soil, for the same distance, resists that shortening.

The resulting force system represents the state of equilibrium between the work of the elastic deformation of the pile and the work of the elastic and plastic deformation of the soil. The load distribution for this depth begins at 0 at the surface, goes quickly to a value of p_1 per square foot, and continues at this almost constant value to the lower end of depth L_1, when it decreases again to practically 0.

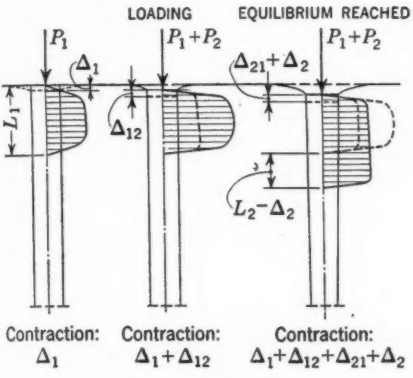
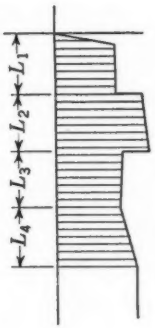


FIG. 15



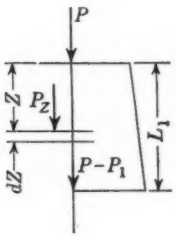


FIG. 16

The load is now increased from P_1 to a total of $P_1 + P_2$. Immediately the equilibrium between the pile and the soil is disturbed, the length L_1 is shortened to a total of $\Delta_1 + \Delta_{12}$, corresponding to the new superposed load. The hydrodynamic stresses in the soil, likewise, including the water stresses within the voids of the soil, are reduced due to the escape of water, in a period varying from a few hours or days, so that the equilibrium is further disturbed because of this condition. To reestablish equilibrium for the new load, it is necessary to add the work done by the soil surrounding the length L_2 of the pile. The final settlement for the total load corresponds to a shortening of length L_2 by Δ_2, plus a supplementary shortening of L_1 by Δ_{21}, due to the diminishing load distribution in length L_1. The load distribution, through $L_1 + L_2$, is balanced according to the same law as that applying to L_1; that is, it changes to a value greater than p_1. This sequence continues for successive increments of load increases, until $P_t = \Sigma P_i$ has made use of the entire resisting power of the pile to the tip, where $\Sigma L_i = L$, the total length of the pile. Only then will the contact between the tip of the pile and the soil have any effect, and only then will the pile settle under its applied working load.

If a load, P_c, traverses the pile for a depth, L_c, without encountering any resistance from the soil surrounding the pile in that distance, P_c can be called a free, or almost free, circulating load. This is the condition that occurs in the case of inconsistent soil (mud, etc.).

It should be borne in mind that the settlement of a pile head may be due, in addition to the shortening of the pile, to the consolidation of the soil about the pile, as well as the settlement of the pile through the soil, after the applied load has passed its critical value. The complexity of the load distribution during a test period depends upon the nature of the soil, and the rapidity with

which test loads are applied or released. By using greater loads, P_i, a dynamic coefficient becomes involved, which, in turn, imposes a load distribution of its own, down along the pile.

After the load has begun to exert its influence at the tip of the pile, the load distribution is further readjusted, and then the pile continues to settle until the critical load is reached. If the load is transmitted to the pile by a jack, the pile descends in jerks, with every stroke of the pump.

Determination of the Elastic Contraction of a Pile.—Consider a pile of total length, L, divided into segments, each transmitting a certain part of the external load on the pile, to its adjacent soil segments, in accordance with a linear law (see Fig. 16). Let the external load equal P and the part transmitted to the soil through the length, L_1, equal P_1. Let P_z be the load at the depth, L_{1z}; then

$$P_z = P - \int_0^Z \left[1 + \frac{L_1 - Z}{L_1}(R_{c1} - 1)\right] \frac{2 P_1 dZ}{L_1(R_{c1} + 1)} \quad \ldots\ldots (7a)$$

and the contraction of the length, L_1, of the pile is

$$\Delta_1 = \int_0^{L_1} \frac{P - \dfrac{2 P_1}{L_1(R_{c1} + 1)}\left[Z R_{c1} - \dfrac{Z^2}{2 L_1}(R_{c1} - 1)\right]}{A_z E} dL' \quad \ldots\ldots (7b)$$

in which E = the modulus in the elasticity of the pile, and the cross section area at the point Z is

$$A_z = \pi \left[\frac{D_b}{2} - \frac{Z(D_b - D_t)}{2 L_1}\right]^2 \quad \ldots\ldots\ldots\ldots (8)$$

where D_b and D_t = the pile diameters at the butt and tip, respectively. The total contraction of the pile is given by a summation of the contractions, Δ_1; thus:

$$\Delta_{\text{total}} = \sum_{i=1}^{i=n} \Delta_i = \sum_{i=1}^{i=n} \int_0^{L_i} \times \frac{P - \sum_{i=1}^{i=i} P_{i-1} - \dfrac{2 P_i}{L_i(R_{ci} + 1)}\left[Z_i R_{ci} - \dfrac{Z^2_i}{2 L_i}(R_{ci} - 1)\right]}{A_{zi} E} dZ_i \quad \ldots\ldots (9)$$

In Eq. 9, P_0 equals 0, Z_i equals 0 for the butt end of L_i, and Z_i equals L_i for the tip end of L_i. If the soil surrounding the pile has one characteristic, L_i can be taken, for practical purposes, equal to 0; that is, when $i > 1$.

For a cylindrical pile with parallel sides, the total contraction is

$$\Delta = \sum_{i=1}^{i=n} \left[\frac{\left(P - \sum_{i=1}^{i=i} P_{i-1}\right) L_i}{E A} - \frac{2 P_i (R_{ci} - 1) L_i}{3 (R_{ci} + 1) E A}\right] \quad \ldots\ldots (10)$$

and when n equals 1:

$$\Delta = \frac{P L}{E A}\left[\frac{2}{3} - \frac{R_c}{3 (R_c + 1)}\right] \quad \ldots\ldots\ldots\ldots (11)$$

Typical solutions of Eq. 11 yield:

R_c	Δ
Free circulating load	$\frac{P\,L}{E\,A}$
0	$\frac{2\,P\,L}{3\,E\,A}$
1	$\frac{P\,L}{2\,E\,A}$
1.5	$\frac{7\,P\,L}{15\,E\,A}$
∞	$\frac{P\,L}{3\,E\,A}$

In the case of a tapered pile, the total contraction is

$$\Delta_{\text{total}} = \sum_{i=1}^{i=n} \left\{ \frac{\left(P - \sum_{i=1}^{i=i} P_{i-1}\right) L_i}{\pi\, E\, \frac{D_{ti}\, D_{bi}}{4}} - \frac{2\, P_i\, R_{ci}\, L_i \left(\frac{D_{bi}}{D_{ti}} - 1 - \log_e \frac{D_{bi}}{D_{ti}}\right)}{\pi\, E\, (R_{ci}+1) \left(\frac{D_{bi} - D_{ti}}{2}\right)^2} \right.$$

$$\left. - \frac{P_i\, (R_{ci} - 1)\, L_i}{\pi\, E\, (R_{ci}+1) \left(\frac{D_{bi} - D_{ti}}{2}\right)^3} \left[\frac{D_{ti}}{2} - \frac{\left(\frac{D_{bi}}{2}\right)^2}{\frac{D_{ti}}{2}} + 2\,\frac{D_{bi}}{2} \log_e \frac{D_{bi}}{D_{ti}} \right] \right\} \ldots (12)$$

and for $n = 1$, and $D_t = \frac{D_b}{m}$, the contraction becomes

$$\Delta = \frac{A(m) + B(m)\, R_c}{1 + R_c} \times \frac{P\, L}{E\, A_b} \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

in which:

$$A(m) = \frac{m\,(-3\,m^2 + 4\,m - 1 + 2\,m^2 \log_e m)}{(m-1)^3} \ldots\ldots\ldots (14a)$$

and

$$B(m) = \frac{m\,(m^2 - 1 - 2\,m \log_e m)}{(m-1)^3} \ldots\ldots\ldots\ldots\ldots (14b)$$

The solution of these equations for representative values of m is presented in Table 13. For a given length, L_c, the load P_c is a traversing or a free, circulating load, as stated previously, and the contraction is expressed by

$$\Delta_c = \frac{m_c\, P_c\, L_c}{E\, A_{bc}} \ldots\ldots\ldots (15)$$

TABLE 13.—VALUES OF $A(m)$ AND $B(m)$

m	$A(m)$	$B(m)$
1.000	0.6666	0.3333
1.100	0.7157	0.349
1.200	0.7628	0.364
1.300	0.810	0.377
1.400	0.853	0.391
1.500	0.896	0.403
1.600	0.936	0.415
1.700	0.975	0.426
1.800	1.016	0.436
1.900	1.054	0.447
2.000	1.0896	0.456

Analysis of a Loading Test Considering the Elastic Behavior of the Pile.—The part of a test diagram, which is usually neglected—the beginning of the test—is the most valuable for determining the load distribution in the soil

adjacent to a pile. From the five tests presented in Table 2, test (*d*) is useless for this analysis because part of the readings are incorrect, the pile not being plumb. Test (*e*) is also useless for this analysis because the first reading was taken at 80 tons. It should be emphasized that the accuracy of bench-mark readings, using a standard construction level, is generally not sufficient to determine the elastic behavior of a pile. This error may be as high as ±0.001 ft. Pile T20, in Table 2, affords a good illustration of the load distribution during the loading period.

In analyzing Pile T20, the author has assumed that the load is transmitted to the soil through the entire length of the pile, following a linear law. Let $D_b = 17.50$ in., $D_t = 8.00$ in., and $m = 2.19$; then by Eq. 13, the elastic contraction is

$$\Delta = \frac{1.158 + 0.471\,R_c}{1 + R_c} \times \frac{P\,L}{E\,A_b} \quad \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

in which $E = 1{,}400{,}000$ lb per sq in. (if E is greater, R_c is smaller); $A_b = 240.5$ sq in., and $L = 85$ ft. The values of R_c computed from Eq. 16 for test readings of Δ and P, are given in Table 14, the last column of which indicates values of R_c, for $E = 1{,}600{,}000$ lb per sq in. The negative value for R_c in Table 14, under the 15-ton load, indicates that no load distribution can be found which will transmit the load to the soil through the full length of the pile. For a load of $P = 25$ tons, a value of R_c equal to 170.0 indicates that the load transmitted to the soil is 1,170 lb per ft at the butt of the pile, and 6.89 lb per ft at the tip; under a load of $P = 25$ tons, there is practically no load transmission at the lower part of the pile, inasmuch as Δ equals 0.006 ft, which corresponds to an ultimate condition of rest. Therefore, high values of R_c, and even the value $R_c = 8.55$, for the case of $E = 1{,}600{,}000$, are impossible, even as a first approximation in the case of actual soil conditions. On the basis of Table 14, and of the intermediate reading given in Table 2, it can be stated as a practical matter that, for $E = 1{,}400{,}000$ lb per sq in., the full length of the pile does not transmit load to the soil until the load reaches 70 tons; for $E = 1{,}600{,}000$ lb per sq in., the full length of the pile does not act until $P = 50$ tons.

TABLE 14.—SOLUTIONS OF EQ. 16 TO DETERMINE R_c FOR PILE T20

P (tons)	Settlement of pile head (ft)	VALUES OF R_c	
		$E = 1{,}400{,}000$	$E = 1{,}600{,}000$
15	0.003	−10.15	−39.10
25	0.006[a]	+170.00	+8.55
30	0.008	11.05	4.20
50	0.013[b]	14.61	4.82
	0.015	4.58	2.30
	0.016[a]	3.21	1.70[d]
75	0.020[b]	11.05	4.20[c]
	0.024	3.21[c]	1.70[c]
	0.026[a]	2.20[d]	1.19[d]
100	0.028[b]	7.20[c]	3.21[c]
	0.030	4.58[c]	2.30[c]
	0.032	3.21[c]	1.70[c]
	0.045[a]	0.628[d]	0.254[d]

[a] Final. [b] Initial. [c] Includes the buckling effect; that is, the permanent settlement and deformation, if any, caused by the eccentricity of the load and the stiffness of the pile tip embedded in the solid sand layer. The 25-ton increment, however, did not reach the tip of the pile and the equilibrium of the pile-soil system is not yet re-established. [d] Includes the buckling effect.

The settlement readings reported by Mr. Masters were stopped too soon after unloading. Readings should have been continued for a period equal to the loading time (188 hr) or at least the period required for the load of 100 tons

—that is, 120 hr. Had this been done, the final settlement would have been 0.004 ft to 0.008 ft less, corresponding to a maximum elastic contraction of Δ varying from (0.045 − 0.019 =) 0.026 to (0.045 − 0.015 =) 0.030, for a 100-ton load. It seems to the writer that a value of E = 1,600,000 lb per sq in. corresponds more nearly to the actual elasticity modulus, unless the residual settlement of the pile head was actually less than could be estimated from the paper.

The load distribution for pile T20, with P less than 70 tons (or less than 50 tons), begins with a zero distribution at the ground surface, increases rapidly to a value of P_1 per square foot, which corresponds to a value of R_c very much less than 1. Then it increases gradually to the lower part of the effective length of the pile, where the value of R_{c2} varies from about m to 2, and finally decreases to zero. Static equilibrium is considered to be established, or nearly so. Eq. 16 for this condition was solved by determining the working length, L_w, corresponding to the load P, the results being tabulated in Table 15. For this

TABLE 15.—SOLUTION OF EQ. 16, FOR VARIOUS EXTERNAL LOADS

Load P (tons)	Modulus of elasticity E (lb per sq in.)	Effective length L_w (ft)[b]	m	$A(m)$	$B(m)$	Final settlement[c] (ft)	R_c
15	1,400,000	50	1.47	0.885	0.399	0.003	0.774
	1,400,000	50	1.47	0.885	0.399	0.0027[d]	1.35
	1,600,000	55	1.54	0.910	0.408	0.003	0.725
	1,600,000	55	1.54	0.910	0.408	0.0027[d]	1.28
25	1,400,000	58	1.59	0.933	0.414	0.006	0.830
	1,400,000	58	1.59	0.933	0.414	0.0057	1.09
	1,600,000	65	1.71	0.980	0.427	0.006	0.950
	1,600,000	65	1.71	0.980	0.427	0.0057	1.23
50	1,400,000	72	1.85	1.036	0.442	0.016	0.935
	1,600,000	81 to 83[a]	2.10	1.125	0.464	0.016	1.30–1.45
65 to 70	1,400,000	81 to 83[a]	2.10	1.125	0.464	0.019–0.023	4.60–2.10

[a] 81 ft to 83 ft corresponds to the level of the sand stratum. [b] L_w is the approximate effective length of the pile distributing load P to the soil through friction. [c] Indicating a state of equilibrium. [d] Comparing the settlements for 15 and 25 tons it is evident that this value (0.0027 ft) should be even smaller.

analysis no information was available except the few readings listed in Table 2. With these data it was necessary to make a few assumptions in order to limit the range of possible R_c-values. For example, it was assumed that when the loading tests were applied, the dynamic stresses due to driving the pile were no longer present. Also, final equilibrium is assumed to have been attained after a few hours of practically no settlement.

To check the load distribution it is advisable to make an unloading test at about 30% to 50% of the failure load of the pile, just before the effective length of the pile coincides with the actual length. This was done partly for pile T10. The settlement of the pile caps should be recorded carefully from the very beginning, and the applied load should be increased slowly, and at regular intervals. In Table 2(*b*), pile T18 indicates that the entire length of the pile is effective in friction, under a load of 10 tons. It appears that the soil becomes

denser below 40 ft. Pile T10, Table 2(*a*), indicates the following characteristics for $E = 1{,}400{,}000$ lb per sq in.:

Load P (tons)	Length L_w (ft)
16.5	35
31.5	45
41.5	55
45	59.5

For concrete piles the settlement can be determined at any point, although usually at midheight will be sufficient. For this purpose a steel pipe 1½ to 2 in. in diameter is embedded in the axial center of the pile to the depth at which settlement is to be observed. A free, movable rod is inserted within the pipe to the point of observation, and it is the top end of this rod that indicates the settlement. This suggests a very simple method for determining more exactly the load distribution in the soil during the progress of the test. More particularly is this true for piles driven in soils of complex and different characteristics. Only by this simple test can some of the problems in soil mechanics ever be solved.

Under the same load, P, the load distribution varies for piles of different total lengths. On a project dealing with timber friction piles, the test specimens should be in lengths varying by 10 ft—40, 50, 60, 70 ft, etc. For concrete cylindrical friction piles—that is, with parallel sides—it is sufficient for specimens to vary in 20-ft lengths—40, 60, 80 ft, etc. A qualitative description of the soil gained by studying borings, which indicates the depth of the different layers of soil, should always be available.

Table 3, Groups of Piles.—Consider a pile with a total length, L_1. The elastic contraction due to an external load, P_1 is Δ_1 and in this case, R_c is equal to R_{c1}, which, as the load approaches the failure load, is proportional to $m = \frac{D_b}{D_t}$. For a group of piles under the same circumstances, tests would show an elastic contraction, Δ_2, per pile, of $\frac{P_{1f}}{P_{gf}}\Delta_1$; the value of the external load, P_1, remaining the same. This is greater than Δ_1 and corresponds to a much smaller value of R_{c2}. In the foregoing P_{1f} is the failure load for individual piles, and P_{gf} is the failure load per pile in the group. This relationship indicates that in group piles the load distribution increases toward the tip of the pile, as may be expected. Test T10 can be compared with Tests T21 to T29. The elastic contraction of the piles at the time of failure for the group is only a little smaller than that for an individual pile at failure; but even more important, if P_1 is small, and corresponds to $L_w \ll L$, then the tests show that the settlement in the group Δ_2 is from 50% to 200% larger than the expected ratio, $\frac{P_{1f}}{P_{gf}}\Delta_1$, if L_w would have equaled L.

In the two tests, P_{1f} equals 96.5 tons, and P_{gf} equals 55.3 tons. A comparison of predicted and actual measured contraction is given in Table 16.

In Table 3(*a*), Group T21–T29, the effective length, L_w, is equal to the total length, L, for all loads beginning at 28 tons.

Table 4, Skin Friction of Individual Piles.—The differences in the average skin friction between concrete piles with parallel sides and tapered timber piles is less than revealed by Table 4. By adding the weight of the concrete piles (28 tons to 63 tons) the average skin friction of the latter increases to 731 lb per sq ft.

TABLE 16.—COMPARISON OF PREDICTED AND MEASURED CONTRACTIONS

Load P_1 (tons)	Contraction Δ_1 (ft)	Contraction Δ_2 (ft)	
		Predicted	Measured
16.5	0.002	0.035	0.095[a]
31.5	0.006	0.0105	0.0163[b]

[a] Increase of 170% or more. [b] Increase of 55% or more; includes non-elastic settlement.

For the tapered piles the average skin friction at the time of failure is 865 lb per sq ft (Tests T1, in which the pile was not plumbed, and T20, which was not tested to failure, were discarded from this average). The point resistance of timber piles and of concrete piles with larger tip surface was neglected. The difference between 731 and 865 corresponds to the effect of wedging action on the tapered piles. The swelling of wooden piles in the ground might also have been a factor in producing an increase of the critical load compared to that of concrete piles.

Friction Pile Theory.—The author's interpretation of test results is somewhat open to question. This entire problem is concerned with the distribution of the stresses in the soil as produced by a variably located point load in the soil. As early as 1848 Lord Kelvin[12] offered a solution of the stresses due to a point load in an indefinitely extended solid. Later Boussinesq gave his well-known solution of stresses in a semi-infinite body due to forces in the boundary.[13]

More recently, in 1936, Professor Mindlin, using the Galerkin vector, offered a solution for the distribution of stresses in a semi-infinite body, with a point load at any place in the body.[2] The latter solution was the connection between Lord Kelvin's and Boussinesq's problems. Each of these three theories is rigorously valid only for the theoretical, elastic and isotropic bodies (those that follow Hooke's law and which permit the superposition of small strains and stresses). It has been found that the Boussinesq theoretical distribution of stresses does not check for all soils by any means, and especially for cohesionless, or nearly cohesionless, bodies. In 1933 Froehlich introduced a coefficient ν (which he calls a concentration factor) varying from 3 to 6. In the Boussinesq solution ν equals 3 corresponds with a Poisson ratio of 0.5. Froehlich still adheres to the validity of the linear law.[14]

As the name indicates, the coefficient ν helps take into consideration the fact that the modulus of elasticity of certain soils, at various depths, is not constant. It also helps to concentrate stresses more around the point loads, corresponding to the difference between the hypothetical bodies used in the

[12] *Cambridge and Dublin Mathematical Journal*, Vol. 3, 2d Series, 1848, p. 88.
[13] *Application des potentiels*, Paris, 1885.
[14] "Druckverteilung im Baugrunde," Dr.-Ing. O. K. Froehlich.

Lord Kelvin, Boussinesq, and Mindlin stress distribution, and the real soil. Consider a point at O in the soil, and a pile driven not far from it (see Fig. 17). It is clear that the influence of the upper part of the pile, not far from the ground surface, would follow theoretically the Boussinesq law. At distances farther from the ground surface the distribution would approach more nearly that of

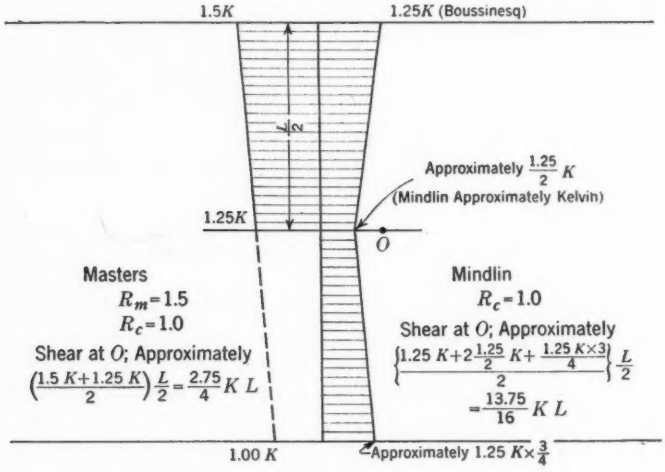


FIG. 17.—VERTICAL SHEAR AT O, MID-HEIGHT (SCHEMATIC)
Mindlin Values Approximately 25% Larger for $R_c = 1$

Lord Kelvin's theory. Since the Kelvin distribution has an effect about half as great as the Boussinesq distribution in the horizontal plane containing point O, it is clear that by using a coefficient, R_m, the stress value at the point O can be determined with as good an approximation as may be desired. For example, if it is assumed that the pile distributes the load to the soil at a constant rate per linear foot, and if the Boussinesq formula is applied to the calculation of the stresses, giving each elementary load a coefficient dictated by a certain rule, then by integration, the stresses can be approximated as accurately as may be desired to the actual stresses. This is true, even if the influence of the point loads below the plane of point O is neglected. If the load distribution is not constant, a variable coefficient, R_m, and a load distribution, R_c, can be introduced. It is in this sense that the coefficient, R_c, used by Mr. Masters, is to be interpreted as corresponding to the coefficient, R_m, of the present discussion, and not as a load distribution along the depth of the piles.

Consequently, if point O is selected at the midheight horizontal plane of the piles, a value of R_m can be obtained which will yield the stresses in that plane. Another value of R_m would give stresses at another point O at a different depth. This is why the Mindlin stress distribution yields tensile stresses independent of the cohesion factor of the soil, and also why the shearing stresses computed by Professor Mindlin's method are higher than those in actual soil. In the future it may be possible to take these two factors into consideration by introducing the cohesion coefficient and the internal friction angle of the soil.

This will be a difficult task, however. With the foregoing explanation in mind, the coefficient R_c in the paper should actually be called R_m to differentiate it from the actual R_c, which is the ratio of the load delivered per linear foot of pile at the top of the pile to the load delivered per linear foot at the bottom of the pile. When this has been done, vertical shears at midheight of friction piles (corresponding to an average of vertical shears for the full length of piles) and safe loads may be computed in practice as indicated in the paper.

Approximate Formula for Vertical Shear in a Pile Group.—Table 6 for the determination of the interaction of piles in a group can also be used by considering the vertical shears as inversely proportional to the center-to-center distances between the piles, in which case the error would not exceed 3%. In the case of pile A—

$$s_{vA} = \frac{P}{2\,\pi\,r_1\,L} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

equals 0.005780 at the midheight of the pile. An even better approximation can be obtained by assuming that the influence of a distant pile A_n on pile A_1 is inversely proportional to $\frac{r_{1n}\,(L^2 + 4\,r^2{}_{1n})^{1.5}}{r_1\,(L^2 + 4\,r^2{}_1)^{1.5}}$, in which r_{1n} equals the distance, $A_1\,A_n$, r_1 = radius of pile A_1 at midheight. This value is then multiplied by the sine of the angle of the line of action. The foregoing proportion is obtained from Eq. 2*c* by neglecting the term with the coefficient $R_c - 1$, and letting Z equal $\frac{L}{2}$; then,

$$s_v = \frac{P\,L^2}{2\,\pi\,r\,(L^2 + 4\,r^2)^{1.5}} \ldots\ldots\ldots\ldots\ldots\ldots (18)$$

In Eq. 18, R_c from the first part of Eq. 2*c* disappears, as was to be expected for Z equals $\frac{L}{2}$ with a linear load distribution. By using Eq. 18 instead of Eq. 2*c* the error is less than 1%, and for the practicing engineer it is more convenient. The vertical pressure beneath the group of piles can be computed as indicated in the paper. In this case again, Eq. 4*b* can be simplified for the plane through the tip of the pile, as follows:

$$\phi_v = \frac{2\,P}{\pi\,L\,r\,(R_m + 1)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

This gives a very good approximation (R_m from Eq. 19 has the same value as R_c from Eq. 4*b*.

In conclusion, it may be stated that by using a coefficient R_m, a better approximation of the stress distribution around the pile corresponding to the true distribution in the soil can be obtained. A better insight into the true behavior of soil under the forces exerted by a friction pile can be obtained only by actual tests.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

DESIGN OF ST. GEORGES TIED ARCH SPAN

Discussion

BY CARLOS A. BEJARANO, JUN. AM. SOC. C. E.

CARLOS A. BEJARANO,[9] JUN. AM. SOC. C. E.[9a]—The derivation of the method of analysis of a novel type of arch has been derived most interestingly and soundly by Professor Garrelts.

In arriving at a simplified expression for the distribution of moments between the arch rib and tie girder, the last three terms of Eq. 12 have been omitted, assuming that the error produced thereby is insignificant. The amount of error introduced by this step can be approximated by assuming reasonable limits to the respective factors, but the exact difference may be calculated after the stress analysis has been completed, using the actual computed values for each factor. This condition may be investigated at panel point 4, where the effect is probably the greatest, since it is close to the quarter point of the span where the stresses approach a maximum.

The radius of curvature of the span may be determined by considering the arch rib in the form of a parabola. The vertical deflections may be computed from the moment influence coefficients, applying the principle of virtual work. Using the computed values for both the deflection and radius of curvature, the last three terms of Eq. 12 will then assume the following values: $-\cos\alpha\,\frac{v}{\rho^2}$ $= -0.00000173$; $\frac{v}{\rho^2}\frac{d\rho}{dl^2}\sin\alpha = 0.00000179$; and $-2\,\frac{dv}{dl}\frac{1}{\rho}\sin\alpha = -0.00000185$. It may be observed that each of these three terms is of the same magnitude, but inasmuch as one of them is of opposite sign, practically the total error will result from either of the other terms. The algebraic sum of the three terms gives -0.00000179.

In order to obtain the precision of the computation, the term $\cos\alpha\,\frac{d^2v}{dl^2}$ may be taken equal to $\frac{d^2\eta}{dl^2}$, which in turn is equivalent to $\pm\frac{M}{E\,I_R}$. For maximum

NOTE.—This paper by J. M. Garrelts, Assoc. M. Am. Soc. C. E., was published in December, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: February, 1942, by R. W. Abbett, M. Am. Soc. C. E.; April, 1942, by Jacob Karol, Esq.; and May, 1942, by Alexander Dodge, Esq.

[9] Parsons, Klapp, Brinckerhoff & Douglas, New York, N. Y.

[9a] Received by the Secretary May 14, 1942.

positive moment the error is − 3.1%, and for maximum negative moment, − 3.4%. Since this is probably the largest error involved in Eq. 13, and also since it is of negative sign, the simplified procedure of analysis is therefore justified.

The question may arise as to whether this will be the amount of error involved wherever this method of analysis may be applied. The foregoing values apply specifically to the St. Georges Tied Arch, but they are a fairly good indication for any other similar structure since none of the factors in Eq. 12, except the curvature, will vary greatly in different arches. Assuming, then, that the third and fourth terms cancel each other and that the total error lies in the omission of the last, one may observe that its magnitude is inversely proportional to the radius of curvature. Thus, the precision is governed by the shape of the arch, and it can be quite accurately determined for any other arch by comparing its curvature with the St. Georges Arch.

Under the heading "The Final Design," Professor Garrelts has noted the influence values for H, in which he has included the effects of rib shortening and tie extension, as indicated in Eq. 17. Actually these latter considerations produce a very minor effect on the result, and it would have been justifiable to omit them from the computations, simplifying the work to some extent. The following is a comparison of the values of H with and without the effects of direct stress:

Load at panel point	Values of H	
	With	Without
1	0.2358	0.2380
2	0.4595	0.4639
3	0.6625	0.6687
4	0.8372	0.8452
5	0.9772	0.9865
6	1.0772	1.0874
7	1.1300	1.1408

Professor Garrelts has suggested an alternate method of analyzing the arch by using a transformed section for the rib, instead of for the tie, which would resist all the bending moment and compression while the tie would act only as a tension member. This would be more in conformance with the regular procedure in the design of an ordinary tied arch, and for this particular reason it probably would be preferable to the original scheme. The transformed moment of inertia for the rib would be $I'_{tr} = I_R + I_G \sec \alpha$. When the total bending was transferred to the tie girder, it was resisted by a transformed section whose moment of inertia was $I_{tr} = I_G + I_R \cos \alpha$. The geometric relation that exists between the two transformed sections is quite apparent. Therefore the preliminary analysis yielding the redundant reactions would be similar in each case. However, in the analysis of the loading conditions to produce maximum fiber stresses at any given point, the two methods will not necessarily yield the same results. The maximum fiber stress is the combined effect of the thrust, moment, and shear due to any given loading. Thus, for maximum total fiber stress, the moment must be taken about the kern points

of each section, rather than through the apparent neutral axis. The additional moment ($\pm \Delta M$) is a function of the properties of the section in question and may be applied as a correction to the moment influence coefficients in determining the loading conditions for maximum stress at any particular point.

If the moment of inertia of the rib differs from that of the tie, the value of ΔM for the larger section obviously will be greater than for the smaller section. The effect of this difference may be noticeable at those panel points where a reversal of stress has taken place; that is, where the bending moment has changed from positive to negative (or vice versa) under a given loading. This may be observed from the moment influence coefficients shown in Fig. 5. For example, notice the effects in the determination of the maximum positive stress at panel point 1. The moment influence coefficients for point 1 when the unit load is at 6 and at 7 are, respectively, + 0.49 and − 2.69. The additional moment at 7 on considering the kern point of the arch rib is 1.00 and, when the kern point of the tie girder is considered, ΔM is equal to 3.11, which is sufficient to produce a positive moment at point 1. In the latter case, then, an additional panel would be loaded to produce maximum positive fiber stress at point 1.

The number of times that this condition might occur depends on the particular characteristics of the structure. It may be noticed that the difference in each case will involve either the loading or unloading of one panel—that is, a small fraction of the live load, which will not produce a very large discrepancy.

To reduce the errors to a minimum, it would be advisable to make an analysis of that transformed structure which more nearly approximates the true behavior of the bridge.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

ENERGY LOSS AT THE BASE OF A FREE OVERFALL

Discussion

BY MESSRS. L. STANDISH HALL, AND HUNTER ROUSE

L. STANDISH HALL,[19] M. AM. SOC. C. E.[19a]—The studies by the author have added needed experimental data to the hydraulics of the energy loss at the base of a free overfall. Engineering literature has been notably lacking in definite information concerning these losses. Experiments of the type described in this paper are necessary to permit the design of safe types of stilling pools at the base of dams or other overfalls.

The determination of values of ϕ in Eq. 2 is particularly interesting, as it supplies data previously lacking, as stated by the author. A graph of the relation between $\frac{h}{d_c}$ and $\frac{d_1}{d_c}$ as expressed in Eq. 4 has not been supplied. Such a graph is helpful in showing that the ratio $\frac{d_1}{d_c}$ decreases rather slowly for values of $\frac{h}{d_c}$ greater than 3.

The author indicates that the velocity head determined from the mean velocity was not of sufficient accuracy, and he has determined the velocity head by the summation of the kinetic energy in the cross section. However, no factor has been added in the basic formulas for this correction either in the energy or the momentum equations. These corrections, investigated by G. G. Coriolis, A. J. C. Barré de Saint-Venant, and others,[20] require the multiplication of the average velocity head by a factor,

$$\alpha = \frac{\int V^3 \, dA}{V^3 A} \qquad (33a)$$

NOTE.—This paper by Walter L. Moore, Jun. Am. Soc. C. E., was published in November, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1942, by Merit P. White, Assoc. M. Am. Soc. C. E.; and April, 1942, by Messrs. Boris A. Bakhmeteff and N. V. Feodoroff, Carl E. Kindsvater, and J. E. Christiansen.

[19] Hydr. Engr., East Bay Municipal Utility Dist., Oakland, Calif.

[19a] Received by the Secretary April 13, 1942.

[20] *Annales des Ponts et Chaussées*, 1836; see also, *Transactions*, A.S.M.E., Vol. 54, 1932, HYD-54-4, p. 57

and a similar factor for the momentum equation,

$$\alpha' = \frac{\int V^2\, dA}{V^2\, A} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (33b)$$

Introducing these factors would alter Eqs. 8 and 9 as follows:

$$\left(\frac{d_f}{d_c}\right)^2 = \frac{d_1}{d_c} + 2\,\alpha'\,\frac{d_c}{d_1} - (2\,\alpha' + 1) \ldots\ldots\ldots\ldots (34a)$$

and

$$\frac{E_1}{d_c} = \frac{d_1}{d_c} + \frac{\alpha}{2}\left(\frac{d_c}{d_1}\right)^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (34b)$$

The values of α and α' determined from the experiments would be of interest. The use of these factors adds to the complex problem of solving Eqs. 34, and their omission is usually allowed for by a factor of safety in the design.

The profiles in Fig. 13 indicate that the jump below a free overfall is about 20% longer than determined by Messrs. Bakhmeteff and Matzke downstream from a regulating sluice.[7] It is possible that the air entrained in the water by the overfall caused the phenomenon noted. This condition has been observed at the toe of many chute structures and, acting upon the writer's suggestion, laboratory experiments were made of the effect of entrained air upon the hydraulic jump below a sluice-gate by introducing air in the bottom of the flume just as the thin sheet of water emerged at high velocity from the sluice.[21] The following conclusions were reached:

(1) The admittance of air into the water before the jump increases the velocity instead of the initial depth.

(2) The increase in velocity due to the entrainment of air, therefore, would tend to require a longer apron below a spillway or a longer stilling basin below a chute.

(3) For values of λ less than 12 the admission of air into the hydraulic jump decreased the comparative length of the jump, whereas for values of λ above 12 the comparative length of jump was increased. Since in the experiments made by the author the values of λ were greater than 12, the entrainment of air in the water would increase the length of the jump.

A free overfall for studying energy losses is very desirable, but from a practical standpoint this type of overfall is not economical, due to the great length of channel below the drop requiring protection. From Fig. 11 it is seen that values of $\frac{d_2}{d_c}$ normally range from 2.0 to 2.5. Messrs. Bakhmeteff and Matzke have found from their experiments that $\frac{L}{d_2}$ averages about 5.0 for quite a range of values of λ.[22] Multiplying these two ratios, $\frac{L}{d_c}$ would vary from 10.0

[7] "The Hydraulic Jump in Terms of Dynamic Similarity," by Boris A. Bakhmeteff and Arthur E. Matzke, *Transactions*, Am. Soc. C. E., Vol. 101 (1936), p. 630.

[21] "Effect of Entrained Air on the Hydraulic Jump," by Emery H. Willes, thesis submitted to the College of Civ. Eng., Utah State Agricultural College, Logan, Utah, in 1937, in partial fulfilment of the requirements for the degree of Bachelor of Science.

[22] "The Hydraulic Jump in Terms of Dynamic Similarity," by Boris A. Bakhmeteff and Arthur E. Matzke, *Transactions*, Am. Soc. C. E., Vol. 101 (1936), Fig. 6, p. 643.

to 15.0, and, if the distance from the brink of the overfall to the start of the jump is designated as L', then $\frac{L'}{d_c} + \frac{L}{d_c}$ would range from 15.0 to 20.0.

The solution of this difficulty is to shorten the distance below the overfall in which the energy can be dissipated. Professor Bakhmeteff has developed a method by providing either a toe basin or barrier which will cause the nappe to be covered by the tailwater.[23] Whether this condition exists, or that of a free nappe with a repelled jump, depends on the value of the drop (Z_0) between the total energy line and the depth (d_2) below the jump. The relative step $\frac{Z_0}{h}$ is the ratio of the drop to the height of the fall. The use of the ratio $\frac{Z_0}{h}$ in conjunction with $\frac{d_c}{h}$ affords a direct relation between the loss above and below the fall. In the author's presentation this can be determined only by following through several equations and diagrams.

Using the author's nomenclature with Professor Bakhmeteff's treatment, this ratio can be expressed by the formula

$$\frac{Z_0}{h} = 1 + \frac{3}{2}\frac{d_c}{h} - \frac{d_1}{2h}\left[-1 + \sqrt{1 + 16\,\phi^2\left(\frac{h}{d_1} + \frac{3}{2}\frac{d_c}{d_1} - 1\right)}\right] \ldots (35)$$

This equation contains too many variables for direct solution, but it can be solved by means of the author's Eq. 4, which also may be expressed in two different forms:

$$\frac{h}{d_c} = \frac{1}{2\,\phi^2}\left(\frac{d_c}{d_1}\right)^2 + \frac{d_1}{d_c} - \frac{3}{2} \ldots\ldots\ldots\ldots\ldots\ldots (36a)$$

and

$$\frac{h}{d_1} = \frac{1}{2\,\phi^2}\left(\frac{d_c}{d_1}\right)^3 - \frac{3}{2}\frac{d_c}{d_1} + 1 \ldots\ldots\ldots\ldots\ldots\ldots (36b)$$

By using $\frac{d_c}{d_1}$ as an argument, Eq. 36a can be solved for $\frac{h}{d_c}$. Values of ϕ^2 must be known in advance and, as only a relation between ϕ^2 and $\frac{h}{d_c}$ can be obtained from Fig. 7(b), the solution must be by trial and error. A direct solution could be made more laboriously by using the cubic equation (Eq. 4). Since all of the ratios in Eqs. 36 are interdependent, the values of ϕ^2 corresponding to equal values of $\frac{d_c}{d_1}$ from Eq. 36(a) can be substituted in Eq. 36b and values of $\frac{h}{d_1}$ calculated. Using the values of the several ratios and of ϕ^2 corresponding to values of $\frac{d_c}{d_1}$, the ratio $\frac{Z_0}{h}$ then can be computed from Eq. 35.

[23] "Hydraulics of Open Channels," by Boris A. Bakhmeteff, Engineering Societies Monograph, 1932, Chapter XXI (a study of this chapter, particularly the numerical examples, will clarify the following discussion by the writer).

From Fig. 11, values of $\frac{Z_0}{h}$ can be derived more easily, since

$$Z_0 = h + \frac{3}{2} d_c - d_2 \quad \ldots\ldots (37a)$$

Then

$$\frac{Z_0}{d_c} = \frac{h}{d_c} + \frac{3}{2} - \frac{d_2}{d_c} \quad \ldots\ldots (37b)$$

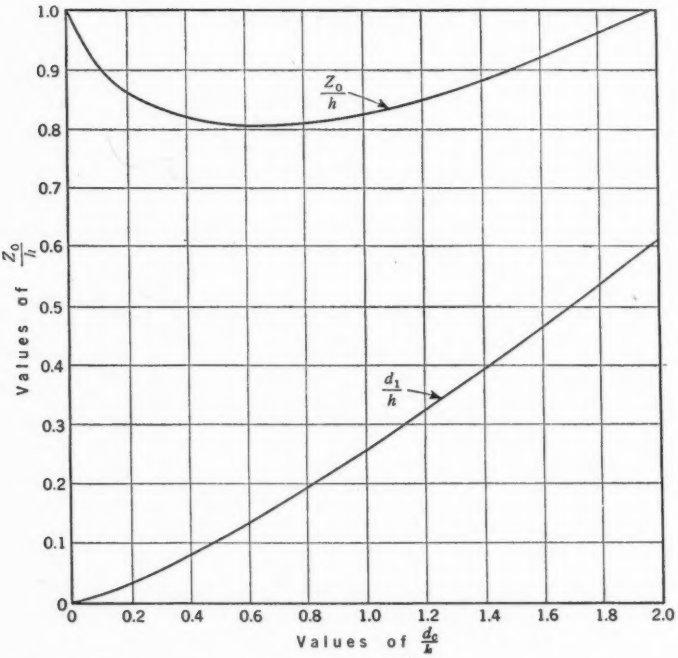


FIG. 27.—CURVES OF RELATIVE WATER SURFACE DROP $\left(\frac{Z_0}{h}\right)$ FOR LIMITING VALUES BETWEEN FREE AND COVERED TOE ROLLS AND RELATION BETWEEN $\frac{d_1}{h}$ AND $\frac{d_c}{h}$ FOR FREE TOE ROLLS BELOW AN OVERFALL

The ratio $\frac{Z_0}{d_c}$ can be computed readily from values taken from curve (3) in Fig. 11; thus,

$$\frac{Z_0}{h} = \frac{Z_0}{d_c} \times \frac{d_c}{h} \quad \ldots\ldots (38)$$

Values of $\frac{Z_0}{h}$ computed from Figs. 7(*b*) and 11 differ slightly, due to the difficulty of scaling accurately from the small scale of the graphs. The relation of $\frac{d_c}{h}$ to $\frac{d_1}{h}$ and $\frac{Z_0}{h}$ is shown in Fig. 27. The flow below the overfall will be submerged

or it will be free, depending on the height of d_2. From Fig. 27, the value of d_1 can be determined first and then that value of Z_0, which separates the covered nappe from the free nappe, can be computed. A toe basin, a barrier, or a combination of both can be provided from these data as may be desired.

Referring to Fig. 28, if a barrier is provided, then

$$Z_0 = h + \frac{3}{2} d_c - d_2 - h_b \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (39)$$

Since all of the values on the right-hand side of the equation are known, or can be assumed, Z_0 can be computed. If the ratio of $\frac{Z_0}{h}$ is less than the value determined from Fig. 27, the nappe will be submerged. If a toe basin is desired, the total height of fall is increased by the amount Δh.

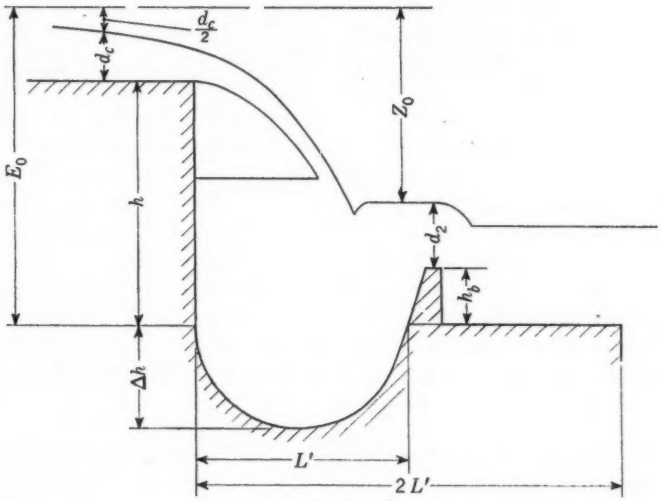


FIG. 28

Let $h' = h + \Delta h$, and assume a value of $\frac{Z_0}{h'}$ and compute h'. The computed value then can be compared with Fig. 27 and adjustments made if necessary. The use of a combination of the barrier and the toe basin usually provides an economy in excavation.

The combination of the toe basin and barrier as shown in Fig. 28 has the advantages of providing a pool in which to dissipate energy and of shortening the length of channel below the overfall requiring protection. This type of structure is not new but has been used effectively for flood and erosion control.[24] However, although the ratio $\frac{Z_0}{h}$ could be computed with reasonable accuracy from existing data, the writer has been unable to find any reference to the

[24] "The Barrier System of Flood Control," by L. M. Winsor, *Civil Engineering*, October, 1938, p. 675.

proper location of the barrier from the brink of the fall, or the distance L' in Fig. 28. This distance can be determined from the equation for the path of a trajectory, traveling at a horizontal velocity, V:

$$X^2 = \frac{2\,V^2}{g}\,Y \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (40)$$

The velocity at the critical depth $V_c = \sqrt{g\,d_c}$, and it may be assumed that the surface velocity V'_c is 20% greater than V_c. Making the additional substitutions of L' for X and $(h + d_c)$ for Y, the length L' becomes

$$L' = C\sqrt{\left(1 + \frac{d_c}{h}\right) d_c\,h} \ldots\ldots\ldots\ldots\ldots\ldots (41)$$

The computed value of C from the assumed surface velocity is 1.7 but, due to the spread of the jet at impact, the value of C should be increased between 1.7 and 2.0. The origin of the parabolic curve of the water surface is upstream from the brink of the drop, as d_0 in Fig. 28 is less than d_c. However, this is not sufficient to compensate for the spread of the stream on impact, and an increase in the value of C is necessary.

The use of longitudinal sills as flow-straightening devices in the stilling pool also would be a very desirable feature when a lateral contraction of the stream exists at the brink of the fall.[25]

Below the barrier the channel should be protected for an additional length that has been arbitrarily made equal to L'. The experiments of the author have made possible the completion of the theoretical treatment of this type of roller basin. Placing the barrier as near the falls as possible eliminates the erosion caused by the undershooting jet, such as would exist if the oversubmergence were created by a barrier located at a considerable distance downstream, as in the author's experiments.

HUNTER ROUSE,[26] ASSOC. M. AM. SOC. C. E.[26a]—Since the writer was privileged to take part in the staff discussions that preceded the experimental study reported upon by Mr. Moore, he has been interested to note the extent to which the final investigation incorporated suggestions made at that time. The dimensionless representation of all data and the adept use of the momentum and energy principles to clarify the general problem are quite in keeping with present-day trends, and Mr. Moore is to be complimented upon an able presentation of his results.

The author evidently restricts his analysis to flow which approaches a drop structure at critical depth—conditions which are commonly considered to define the overfall phenomenon. It is all too seldom realized, however, that flow at critical depth represents only a limiting case of the general overfall problem. In other words, a drop of this nature at the end of a channel will

[25] "Hydraulic Design of Drop Structures for Gully Control," by B. T. Morris and D. C. Johnson, *Proceedings*, Am. Soc. C. E., January, 1942, p. 17.

[26] Prof., Fluid Mechanics, State Univ. of Iowa, Cons. Engr., Iowa Inst. of Hydr. Research, Univ. of Iowa, Iowa City, Iowa.

[26a] Received by the Secretary April 30, 1942.

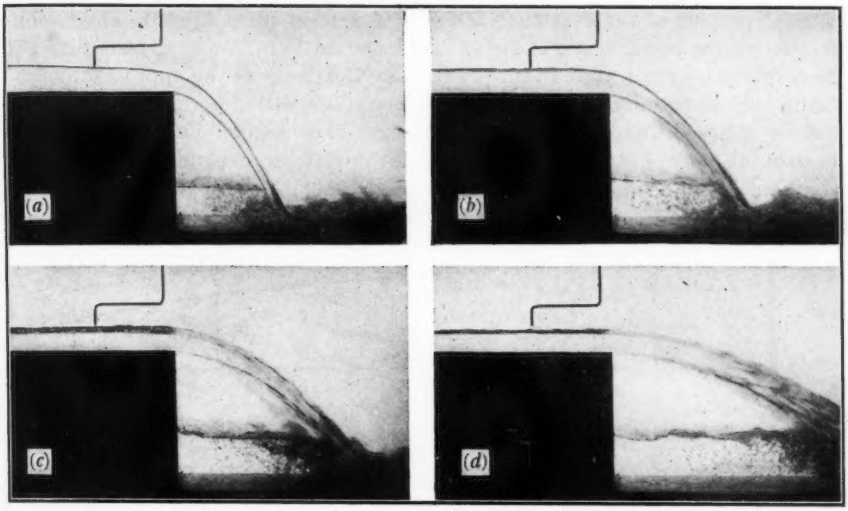


FIG. 29.—NAPPE PROFILES AT FROUDE NUMBERS OF APPROXIMATELY 1, 2.5, 6, AND 14

serve as a channel control only if the channel slope is less than the critical—that at which the rate of discharge in question will produce uniform flow at critical depth. If the channel is steep, the depth of approach will be less than the critical, and the nappe profile and the accompanying momentum and energy

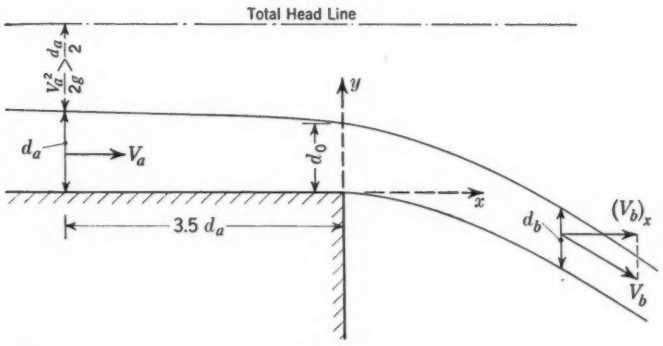


FIG. 30

characteristics will differ accordingly from those in the limiting case studied by Mr. Moore. Since the use of drop structures is not necessarily restricted to channels of mild slope, the importance of a more inclusive study than Mr. Moore's is obvious.

To illustrate this point, the writer makes use of data obtained by A. F. Leder, Prof. J. F. Mangold, Assoc. M. Am. Soc. C. E., and Prof. G. W. Thomas in the Hydraulics Laboratory of the University of Iowa, in Iowa City, as a

special project of the National Defense Course in Mechanics of Fluids conducted during the summer of 1941. As shown in Fig. 29, a drop structure 1 ft high was placed in a glass channel 1 ft wide, the depth of the approaching flow being held essentially constant as the velocity was varied by means of a sluice-gate a short distance upstream. Although the channel floor was actually horizontal, the resulting profiles should not deviate appreciably from those for similar depths and velocities at the end of a channel which is hydraulically steep.

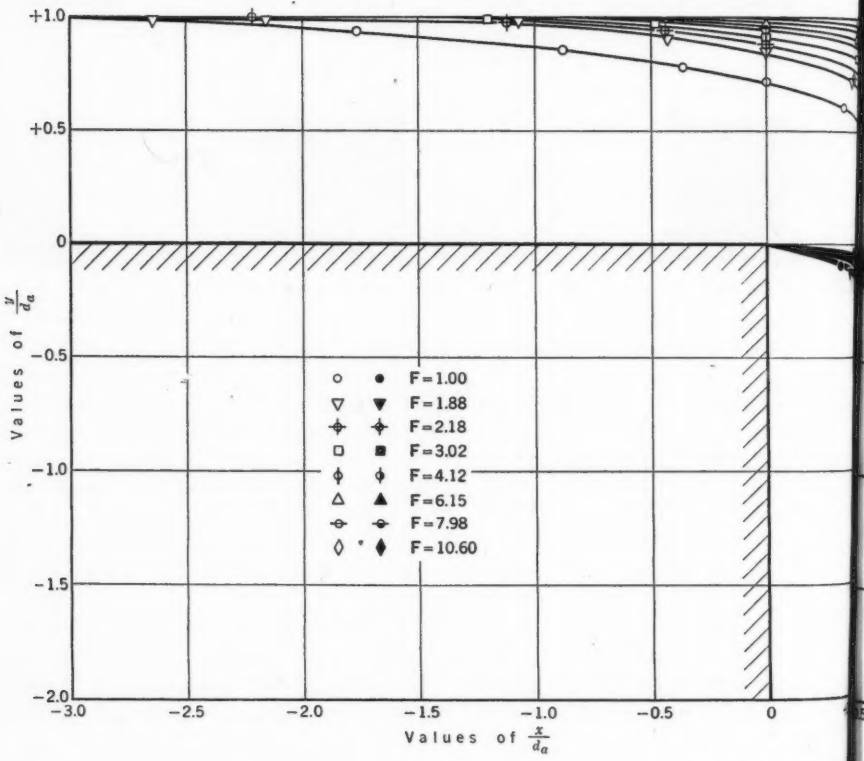


FIG. 31.—PROFILE DATA [illegible] FLOWS

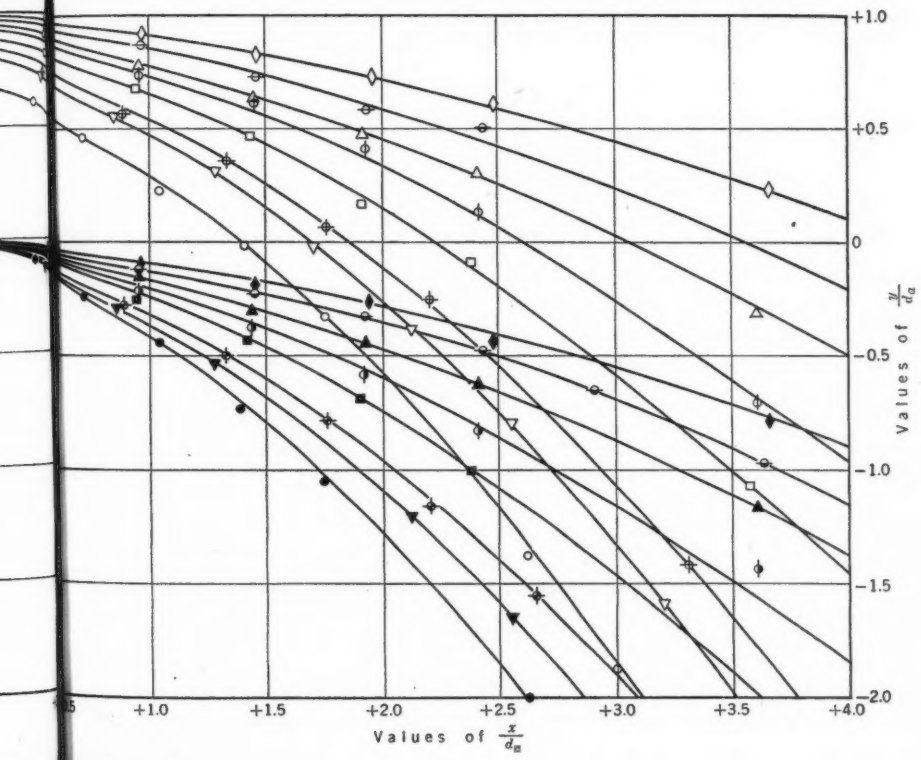
Through use of the momentum principle, it is at once possible to evaluate the ultimate vertical thickness of the nappe, which is approached asymptotically as the pressure intensity within the falling sheet approaches that of the atmosphere. With reference to Fig. 30, the rate of change of the horizontal component of momentum per unit width between sections a and b will be $\rho\{[(V_b)_x]^2\, d_b - (V_a)^2\, d_a\}$, and the force per unit width producing such change will be $\frac{\gamma\,(d_a)^2}{2}$. Equating these values,

$$\frac{\gamma\,(d_a)^2}{2} = \rho\,\{[(V_b)_x]^2\, d_b - (V_a)^2\, d_a\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (42)$$

Upon introducing the continuity relationship $V_a\, d_a = (V_b)_x\, d_b$ and the dimensionless ratio $\mathbf{F} = \frac{(V_a)^2}{g\, d_a}$, it will be found that

$$\frac{d_b}{d_a} = \frac{2\,\mathbf{F}}{1 + 2\,\mathbf{F}} \qquad (43)$$

The quantity **F** will be recognized as the Froude number of the approaching flow, to which Mr. Moore's "kinetic flow factor" is dimensionally equivalent.



ous Froude Numbers

For the particular case **F** = 1, which is that investigated by Mr. Moore, $\frac{d_b}{d_a}$ attains the minimum value $\frac{d_b}{d_a} = \frac{2}{1+2} = 0.667$, which is sometimes erroneously assumed to be the vertical depth at the brink. The brink depth is known to attain the minimum value $d_0 = 0.715\, d_c$ for a Froude number of unity,[15] but its magnitude at higher values has not previously been measured. An analytical study of this function was attempted in 1937 by W. B. Langbein,[27] Assoc. M. Am. Soc. C. E., and, as a matter of fact, the measurements presented

[15] "Discharge Characteristics of the Free Overfall," by Hunter Rouse, *Civil Engineering*, April, 1936, p. 257.

[27] *Civil Engineering*, May, 1937, pp. 349–350; see also July, 1937, p. 518.

in this discussion were made to determine the actual form of the functional relationship. As evaluated for the writer by B. V. Bhoota, Jun. Am. Soc. C. E., the profile data at various Froude numbers are presented in dimensionless form in Fig. 31, the depth d_a at the distance $x = -3.5\,d_a$ from the crest being used as reference. Values of the ratios $\frac{d_0}{d_a}$ and $\frac{d_b}{d_a}$ obtained from these profiles are plotted in Fig. 32 against the corresponding values of the Froude number. Values of $\frac{d_b}{d_a}$ follow closely the general trend of Eq. 43, approaching the limit of unity as the Froude number becomes large—that is, as the gravita-

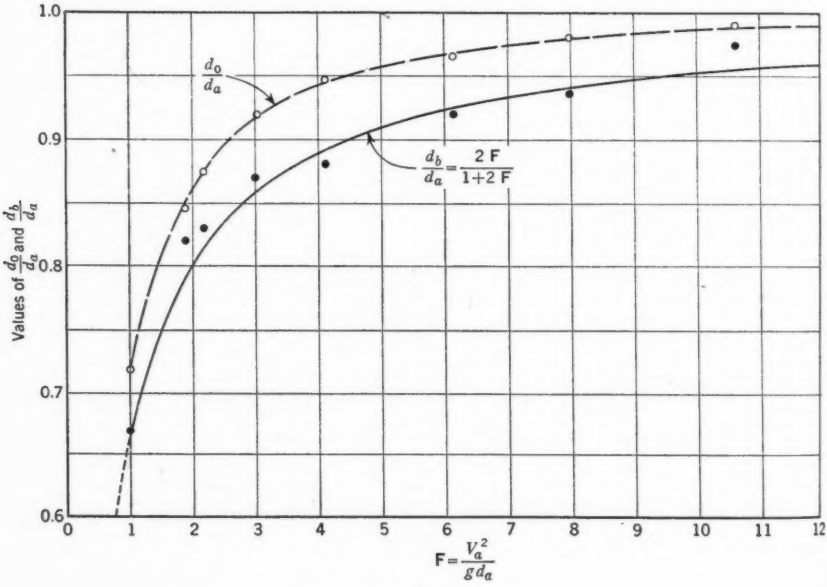


FIG. 32

tional effect upon the nappe form steadily diminishes in relative magnitude. In agreement with earlier measurements by the writer, the ratio $\frac{d_0}{d_a}$ attains the limit 0.715 for a Froude number of unity (Mr. Moore's case); this ratio likewise approaches unity, but somewhat more rapidly, with increasing values of F.

The writer regrets that the backwater depth below the nappe and the subsequent rate of energy dissipation were not measured at the time these studies were made. However, through use of the parameters suggested by the writer, the methods of analysis presented by Mr. Moore could be adapted to all phases of the overfall problem, and the writer recommends their further experimental study to the Pasadena laboratory. Until designers of drop structures become aware of overfall characteristics at Froude numbers in excess of unity, the successful performance of such structures cannot be fully guaranteed.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

MOMENT BALANCE: A SELF-CHECKING ANALYSIS OF RIGIDLY JOINTED FRAMES

Discussion

BY MESSRS. A. A. EREMIN, L. E. GRINTER, AND B. J. ALECK

A. A. EREMIN,[7] ASSOC. M. AM. SOC. C. E.[7a]—Distribution of moments in rigid frames or continuous beams is an interesting problem in the analysis of statically indeterminate structures. In the past few decades numerous analytical and graphical methods of distribution of moments have been developed. The methods are based on the same well-known principle of the deformation of an elastic member loaded with the moment area from the given loading. Therefore, the proximity of the methods is usually the same, and the merit of the methods varies with their simplicity and the saving of time and effort in their application to practice.

The slope-deflection equations for the distribution of moments converge rapidly. Therefore, various attempts have been made to develop algebraic equations that will reduce the number of steps in the computations.

To reduce the balancing operations the author, in Eq. 7, assumes the value of M'. To select a suitable approximation for M' is not always a simple matter. Often it requires preliminary analytical computations or graphical constructions. The assumed value of M' may vary from its exact value not only in magnitude but also in sign. The operations with the assumed value of M' may take more time than the additional steps required to balance the fixed-end moments in the successive approximations.

Considerable difficulty is involved in computing the moment area and its center in Eq. 2. This is especially true in relation to the haunched beams carrying irregular loading.

The graphical method of distribution of moments developed by L. H. Nishkian and D. B. Steinman,[8] Members, Am. Soc. C. E., has similar difficulty

NOTE.—This paper by R. J. Cornish, Esq., was published in May, 1942, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: May, 1942, by Messrs. D. D. Matthews, and William A. Larsen.

[7] Associate Bridge Engr., Bridge Dept., Div. of Highways, State Dept. of Public Works, Sacramento, Calif.

[7a] Received by the Secretary May 4, 1942.

[8] "Moments in Restrained and Continuous Beams by the Method of Conjugate Points," by L. H. Nishkian and D. B. Steinman, *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 1.

in establishing the centers in the moment areas. Complexity in finding the center of moment area in the graphical constructions was an important reason why the graphical method could not find wide application, and preference is generally given to the algebraic methods based on the fixed-end moment distribution. It is quite possible to prepare the tables and diagrams for computing the moment areas and their centers for various shapes of members and type of loading. However, these tables and diagrams will be more complicated than those prepared for computing fixed-end moments.

Distribution of moments in rigid frames or continuous beams has an important economic value in the theory of design. Therefore, the author is to be commended for his contribution toward the improvement of designing methods.

L. E. GRINTER,[9] M. AM. SOC. C. E.[9a]—This interesting and useful paper presents a method of successive approximations for analyzing rigid frames for fixed loadings. It should be noted distinctly that the procedure is one of successive approximations rather than a method of successive corrections. In this feature, it is different from the Cross method of moment distribution which has been emphasized by the writer as a convenient tool of analysis. Nevertheless, there are several points in common between these procedures.

In the author's paper the working formula (Eq. 6) is $M = \frac{M'}{2} - \frac{X}{2} + \frac{\kappa}{2}$, which is derived from the slope-deflection equations. This may lead some readers to misunderstand the background of the method, which is not the use of slopes and deflections as redundants. This will be evident if the working formula is written out without the need for mathematical derivation by a study of the physical action of the structure in Fig. 8. The moment at the end O of

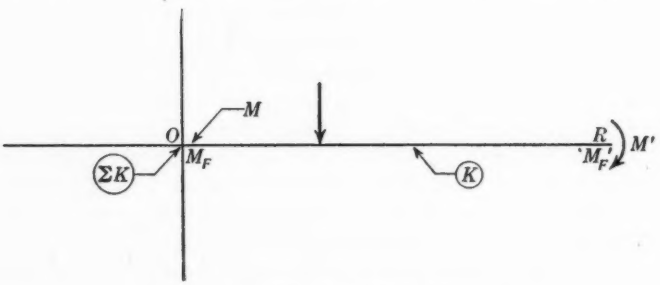


FIG. 8.—DERIVATION OF BASIC FORMULA

the member OR may be obtained as the summation of the carry-over effects at that point of the moments applied at R $\left(\text{that is, } \frac{M'}{2} \text{ and } \frac{M'_F}{2}\right)$ plus the fixed-end moment at the end O plus the balancing moment at O for the member OR when the joint is allowed to rotate. Thus, without reference to sign, one

[9] Vice-Pres. and Dean, Graduate School, Illinois Inst. of Technology, Chicago, Ill.

[9a] Received by the Secretary May 15, 1942.

may write

$$M = \left[\frac{M'}{2}\right] + \left[M_F + \frac{M'_F}{2}\right] + \left[\frac{K}{\Sigma K}\left(\frac{\Sigma M'}{2} + \Sigma M_F + \frac{\Sigma M'_F}{2}\right)\right] \ldots (11)$$

In Eq. 11, the three terms enclosed in brackets will be found to correspond exactly to the author's definitions of the three terms of his Eq. 1. Signs, of course, follow automatically if some such rule is adopted as "a positive moment tends to rotate the adjacent joint clockwise."

After Eq. 6, or Eq. 11, is established, the author's procedure is simple and convenient since it permits the operator to shorten the convergence by exercising his judgment. In many problems, it is true that the final moments can be "guessed" with reasonable accuracy, and these become the first M'-values in Eq. 11. The values of M_F and M'_F, being fixed-end moments, are known exactly. Evidently, all values of M can then be computed by repeated use of Eq. 11. It must be realized, of course, that a given moment is an M-value for that point but an M'-value when the moment at the far end of the member is being computed.

This method of analysis has many points in its favor, particularly if the procedure is brought clearly into view by eliminating the unnecessary terms X and κ. On the other hand, it lacks the close association with the deformations of the structure that is so evident in the Cross procedure. For example, it is convenient and almost necessary to follow the author's procedure of placing the calculations in tabular form, rather than upon the diagram of the structure, since a term such as the author's X or κ is not an individual moment but a combination of a number of moments that are best selected from a well-organized table. The writer tried the other procedure of writing all moment terms upon an outline of the structure but found it not entirely practicable. Also, there were a great many more terms and many more calculations than are involved in an analysis by the usual method of balancing moments.

To be more useful than the regular procedures of balancing moments, this method should be preceded by a careful estimate of the moments. If the moments can be estimated so accurately that only one revision becomes necessary, the method might be faster than any other in common use; but, if the method is for use by inexperienced designers, several revisions of moments will be necessary and the procedure will be rather lengthy.

The self-correcting feature of the method deserves comment. Since this is a method of successive approximations, an error in recording a moment will be corrected in the succeeding revision. In this direction the method has merit. The check of $\Sigma M = 0$ at each joint, of course, is not an independent check because this tool is used as one part of the procedure of analysis. An error in computing a fixed-end moment, in computing K or ΣK, or in sign may not be discovered. In these computations, care must be exercised, as always.

The writer suggests that this method should be given the author's name because the title "Moment Balance" or "Balancing Moments" has been used for years as an alternate to "Moment Distribution" to designate the Cross method, or its variations, based upon successive corrections.

B. J. ALECK,[10] JUN. AM. SOC. C. E.[10a]—"Moment balance" may be extended to apply to members of variable moment of inertia and to some other special cases. Let all moments and joint rotations be considered positive when they are counterclockwise at the ends of the member. Let: $(K_f)_{AB}$ = stiffness at A when B is free to rotate; $(M_f)_{AB}$ = fixed-end moment at A when B is free to rotate; and C_{AB} = carry-over factor from A to B. Then

$$M_{AB} = (M_f)_{AB} + C_{BA} M_{BA} + (K_f)_{AB} \phi_A \ldots\ldots\ldots\ldots (12a)$$

and

$$\Sigma M_{AX} = \Sigma[(M_f)_{AX} + C_{XA} M_{XA}] + \Sigma(K_f)_{AX} \phi_A = 0 \ldots\ldots (12b)$$

at joint A; and

$$\phi_A = -\frac{\Sigma[(M_f)_{AX} + C_{XA} M_{XA}]}{\Sigma(K_f)_{AX}} \ldots\ldots\ldots\ldots\ldots (12c)$$

The procedure from this stage follows the outline set up by Mr. Cornish: $(M_f)_{AX}$ is computed; $C_{XA}\ M_{XA}$ is approximated and used in Eq. 12*c* to find ϕ_A; and ϕ_A is then used in Eq. 12*a* to find the moments at joint A.

It should be emphasized that members with known conditions at the far end can be represented with "modified" stiffness and fixed-end moment, in a manner identical with that used in "moment distribution." For such members,

$$M_{AB} = (M_{MF})_{AB} + (K_M)_{AB} \phi_A \ldots\ldots\ldots\ldots\ldots (13)$$

when $(M_{MF})_{AB}$ = modified fixed-end moment and $(K_M)_{AB}$ = modified stiffness.

Since Eq. 13 is of the same form as Eq. 12*a*, it may be used whenever convenient as a substitute, and relieves one of estimating M_{BA} before M_{AB} is known. The moments at the far end of such members are found, using the modified carry-over factors.

For common members, data compiled for use with the moment-distribution method can be modified to evaluate all the terms used in Eqs. 12 and 13. For uncommon members, they can be evaluated from the properties of the proper $\frac{M}{EI}$-diagram.

Mr. Cornish's method provides a check in one cycle on answers obtained by any method. To the writer, this feature alone seems sufficient reason for engineers to learn the principles of "moment balance."

---

[10] Asst. Structural Engr., War Dept., U. S. Engr. Office, Cincinnati, Ohio.

[10a] Received by the Secretary May 25, 1942.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

THE GREASE PROBLEM IN SEWAGE TREATMENT

Discussion

BY W. S. MAHLIE, ESQ.

W. S. MAHLIE,[12] ESQ.[12a]—For years sanitary engineers have accepted the fact that there is grease in sewage and that it hinders plant operation. There have been a number of instances in which grease was considered a factor in plant design, but by and large it was not given a very intensive study, and the paper, therefore appears at an opportune time. The question of sewage treatment at army cantonments has focused some attention on this grease problem. Considering the large percentage of grease in raw or primary sludge, it is rather surprising that more work has not been done on the effects of grease in sludge digestion. Messrs. Fales and Greeley comment on the lack of a more standardized method of grease determination. The latest edition of "Standard Methods" contains reference to this subject. The authors are to be complimented on their excellent paper, particularly the part dealing with the methods of grease removal.

NOTE.—This paper by Almon L. Fales and Samuel A. Greeley, Members, Am. Soc. C. E., was published in February, 1942, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: February, 1942, by Arthur D. Weston, M. Am. Soc. C. E.; and May, 1942, by Harvey F. Ludwig, Jun. Am. Soc. C. E.

[12] Chemist in Chg. of Water and Sewage Purification, Fort Worth, Tex.

[12a] Received by the Secretary May 22, 1942.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

DESIGN OF SIGN LETTER SIZES

Discussion

BY CHARLES M. NOBLE, M. AM. SOC. C. E.

CHARLES M. NOBLE,[18] M. AM. SOC. C. E.[18a]—Marking the second step in the development of highway sign technique, it is hoped that this paper will stimulate interest in continuing the advance in research in order that complete practical scientific data will be available to the highway and traffic engineer.

The use of adequate signs is becoming increasingly important due to the advancing development of modern highway design that utilizes the principle of free flowing traffic. This principle, involving as it does the elimination of cross traffic and left turns by the use of grade separations with right turns only, results in a type of highway facility that is confusing to the uninitiated motorist in that he is required to turn right when he wishes to go left. Turning right in order to "go left" is contrary to the instincts of the average motorist. Consequently, it is of the utmost importance that directions (the sign) be of such design, size, and clarity that the driver will have ample time to observe, read, react, and maneuver properly. The modern "cloverleaf" is designed on the assumption that the motorist will make the proper turns, and he must do so if he is to avoid losing valuable time and "getting lost." Adequate signing is fundamental, therefore, if the economic value of the expensive grade-separated type of highway is to be fully realized; and the utmost care and thought should be given to the design and placement of highway direction signs.

The authors have presented, step by step, a logical reasoning in an analysis for determinating the correct letter sizes to be used in designing a sign for particular conditions of design speed and site location. The principle of breaking the elements of reading, perception, and reaction into units of time which in turn can be converted into distance and then related to the distance at which various sized letters can be read by the motorist, is fundamentally sound. (Incidentally the table in the paper which presents the visibility distances of letters is based on the eighty percentile group of motorists.)

NOTE.—This paper by Adolphus Mitchell, Assoc. M. Am. Soc. C. E., and T. W. Forbes, Esq., was published in January, 1942, *Proceedings*. Discussion has appeared in *Proceedings*, as follows: May, 1942, by Guy Kelcey, M. Am. Soc. C. E.

[18] Arlington, Va.

[18a] Received by the Secretary May 22, 1942.

A study of the material presented reveals that letters of a much larger size than customarily utilized should be used if the highway sign is to be fully effective. These large letters when assembled together into a sign message will result in a sign panel of such dimensions that it will stagger the designer when compared with ordinary standards. In this connection it is interesting to note that some of the direction signs on the Pennsylvania Turnpike are 14 ft wide by 9 ft high and these signs do not give the impression of being oversize for the conditions. The practice of utilizing letters of different size on the same sign, as outlined by the authors, was adopted successfully on the Turnpike, and it is believed that this is a practical method of keeping the size of the sign panel within reasonable bounds.

The analysis by the authors indicates that a smaller size of direction sign may be utilized if the sign is placed in advance of the turnout rather than at the point of turnout. For example, at a speed of 50 miles per hr an advance sign must be seen for a distance of 530 ft (six-lane highway width) requiring 12-in. series D letters, whereas a sign placed at the point of turnout should be seen a distance of 870 ft requiring 18-in. series D letters.

These required distances lead to a complication when signing urban highways in congested metropolitan areas where turnouts are frequent and often very close together with insufficient distance available to satisfy the requirements of speed and vision. Such conditions will tax the ingenuity of the sign designer.

The authors have covered the subject admirably, having placed a new tool in the hands of the engineer which will add to the effectiveness of the highway as well as to the safety and comfort of the traveling public, and have therefore performed a valuable service.